LLNL-TR-673244

LAWRENCE
LIVERMORE
NATIONAL
LABORATORY

BUILDING 341 Seismic Evaluation

J. Halle

June 15, 2015

Disclaimer

This work performed under the auspices of the U.S. Department of Energy by Lawrence Livermore National Laboratory under Contract DE-AC52-07NA27344.

Degenkolb

LAWRENCE LIVERMORE NATIONAL LABORATORY
LIVERMORE, CALIFORNIA

BUILDING 341
Seismic Evaluation

December 20, 2013

Degenkolb Job Number B3189012.00

Degenkolb Engineers
1300 Clay Street
Oakland, California 94607
www.degenkolb.com

Table of Contents

Table of Contents ii
List of Figures iii
Executive Summary 1
1.0 Introduction 3
2.0 Seismicity and Soils 4
3.0 Building Description 6
3.1 Building Structure 7
3.1.1 Increment 1 .... *7*
3.1.2 Increment 2 .... *11*
3.1.3 Increment 3 .... *12*
3.2 Condition Assessment 14
4.0 Seismic Evaluations of Increments 2 and 3 15
4.1 Increment 2 15
4.2 Increment 3 16
5.0 Seismic Evaluation of Increment 1 18
5.1 Summary of Tier 3 Seismic Evaluation 18
5.2 Results of Tier 3 Seismic Evaluation 19
6.0 Seismic Strengthening 21
Appendix A: Increment 2 Tier 1 Check Lists and Calculations
Appendix B: Increment 3 Tier 1 Check Lists and Calculations
Appendix C: Increment 1 Tier 3 Calculations

List of Figures

Figure 1 – Lawrence Livermore National Laboratory Vicinity Map 4
Figure 2 – Comparison of Response Spectrum 5
Figure 3 – Building 341 Key Plan 6
Figure 4 – 3D Image of Increment 1 7
Figure 5 – Increment 1 Foundation Plan 8
Figure 6 – Increment 1 East-West Cross-Section 9
Figure 7 – Southwest Corner of Mechanical Room 9
Figure 8 – Increment 1 Low Roof Framing Plan 11
Figure 9 – Increment 2 Roof Framing Plan 11
Figure 10 – Increment 2 Tension Only Bracing 12
Figure 11 – Increment 3 Roof Framing Plan 13
Figure 12 – 3D Perspective View of Analysis Model 18
Figure 13 – ASCE 41-13 General Response Spectrum 19
Figure 14 – Low Roof Framing Plan with Seismic Strengthening 22
Figure 15 – Collector Connection at Lines 2/C and 2/E 23
Figure 16 – Collector Connection to Shear Wall at Lines 2/C and 2/E 24
Figure 17 - Collector Connection along Line 2 25
Figure 18 – Collector Connection at Lines 3/C and 3/E 25
Figure 19 – Precast Panel-Panel Connection Strengthening (Interior Face) 26
Figure 20 – Precast Panel-Panel Connection Strengthening (Exterior Face) 26
Figure 21 – Strengthening of Existing Horizontal Bracing at Lines 2/A and 2/K 27
Figure 22 – Partial High Roof Framing Plan - Horizontal Bracing 28
Figure 23 – Increment 2 – East and West Braced Frame Elevations 29
Figure 24 – Strengthening of Tension Only Brace Connection at Roof 29
Figure 25 - Strengthening of Tension Only Brace Connection at Column Base 30

Executive Summary

The Seismic Evaluation of Building 341 located at Lawrence Livermore National Laboratory in Livermore, California has been completed. The subject building consists of a main building, Increment 1, and two smaller additions; Increments 2 and 3.

Increment 1, constructed in 1963, is a one-story steel framed structure with a partial mezzanine, concrete shear walls and exterior precast concrete panels. The building is rectangular in plan with plan dimensions of approximately 180 feet in the North-South direction and 140 feet in the East-West direction. Overall building height varies between 22 feet at the low roof and 32 feet at the high roof.

Increment 2, constructed in 1973, is a one-story steel framed building with exterior precast concrete tilt-up panels. Increment 2 is located on the West elevation of Increment 1 and the east wall of Increment 2 is created by the West wall of Increment 1. There is a two-inch separation between the two structures. The building is rectangular in plan with overall dimensions of 102 feet in the North-South direction and 25 feet in the East-West direction. Building height is approximately 20 feet.

Increment 3, constructed in 1975, is a one-story concrete and wood framed structure. Increment 3 is located on the North elevation of Increment 1 and the south wall of Increment 3 is created by the North wall of Increment 1. Furthermore, a portion of the Increment 3 roof is supported directly by the precast panels on the North elevation of Increment 1. The building is rectangular in plan with overall dimensions of 39 feet in the North-South direction and 76 feet in the East-West direction. Building height is approximately 12 feet.

Tier 1 seismic evaluations of Increments 2 and 3 and a Tier 3 seismic evaluation of Increment 1 were conducted in accordance with ASCE 41-13, with a seismic performance goal of Life Safety, which is in agreement with Performance Category 1 (PC-1) as defined in Department of Energy Standard 1021-1993.

Based on our evaluation the building does not meet a Life Safety performance level for the BSE-1E earthquake ground shaking hazard. The BSE-1E is the recommended seismic hazard level for evaluation of existing structures and is based on a 20% probability of exceedence in 50 years.

Our evaluation identified the following key deficiencies:

Increment 1

- Precast panel to panel connection on lines 4, A and K along the vertical joint are overstressed.
- The roof diaphragm at the high roof along lines E and K is overstressed in shear.
- There is a lack of a collector connection between the low roof framing and the concrete shear walls on lines C and E. There are significant loads that need to be dragged to these walls, yet there is no direct connection.

- There is a lack of a collector connection between the low roof/mechanical roof framing south of line E and the concrete shear wall on line 2.
- The horizontal double angle diaphragm bracing connections are overstressed at lines 2/A and 2/K.

Increment 2

- The braced frames located on the East and West sides of Increment 2 do not have the capacity to develop the yield strength of the braces.
- The gap between Increments 1 and 2 is only 2-inches, which will result in damage due to pounding.
- In addition to relatively small gap between the two structures, the expected lateral drift in the east-west direction exceeds the acceptable limit of 2.5%.
- The moment frame beam-column connections do not have adequate capacity to develop flexural yielding of the beams.

Based on the deficiencies listed above, we recommend that the following seismic strengthening measures be implemented to achieve a Life Safety performance level:

1. Increment 1
    a. Add collector connections between the existing low roof steel beam and the concrete shear walls on lines C and E.
    b. Strengthen the beam-column connections along line 2 at the low roof to transfer the collector demands to the concrete shear wall on line 2.
    c. Strengthen the beam-column connections along lines C and E at the low roof to transfer the collector demands to the concrete shear walls on lines C and E.
    d. Strengthen precast panel to panel connections along vertical joints at lines 4, A and K.
    e. Strengthen double angle horizontal bracing connections at grid lines 2/A and 2/K by adding welds to the existing connections.
    f. Strengthen the high roof diaphragm between lines E and K.
2. Increment 2
    a. Strengthen the braced frame connections on the east and west sides of the building to develop the capacity of the diagonal braces.

1.0 Introduction

This report presents the results of the Tier 1 and Tier 3 Seismic Evaluations of Building 341 located at Lawrence Livermore National Laboratory in Livermore, California. Building 341 consists of three separate structures; a main building (Increment 1) and two small additions (Increments 2 and 3). As outlined in our proposal we performed Tier 1 evaluations on the small additions and a detailed Tier 3 evaluation on the main structure.

The seismic evaluations were performed in accordance with ASCE 41-13 Seismic Evaluation and Retrofit of Existing Buildings. Our evaluations only included the main building structures. We did not evaluate the nonstructural systems or any non-building structures located within the buildings. As requested by Lawrence Livermore National Laboratory the desired performance level is Performance Category 1 (PC-1) as defined in Department of Energy Standard 1021-1993, which corresponds to Life Safety performance (S-3) in accordance with ASCE 41-13.

Our evaluation was based upon the original structural design drawings prepared by the following parties:

- Increment 1: Structural drawings prepared by Garretson, Elmendorf, Klein and Reibin, Architects and Engineers, dated February 18, 1963.
- Increment 2: Architectural and Structural drawings prepared by University of California Lawrence Radiation Laboratory Plant Engineering, dated April 5, 1973.
- Increment 3: Architectural and Structural drawings prepared by Garretson, Elmendorf, Zinov and Reibin, Architects and Engineers, dated October 29, 1975.

Material properties for Increment 1 were based on a previous seismic evaluation report "Preliminary Seismic Evaluation for B-341", dated December 4, 1984.

A site visit was performed on November 6, 2013 to review the existing condition of the building and confirm that the available documents accurately represent the as-built structure. Based on our site visit we were able to confirm that the majority of the lateral load resisting system was built as shown on the original construction drawings.

2.0 Seismicity and Soils

The building site is located near the center of the Lawrence Livermore National Laboratory Campus on the north side of Third Street west of South Gate Drive, as shown in Figure 1. The building site is level, as is the majority of the campus. According to the USGS Quaternary Fault Maps the Greenville Fault is located approximately 3 km to the north east of the campus and the Las Positas Fault is located approximately 1 km to the south. Both of these faults are capable of generating strong ground shaking at the building.

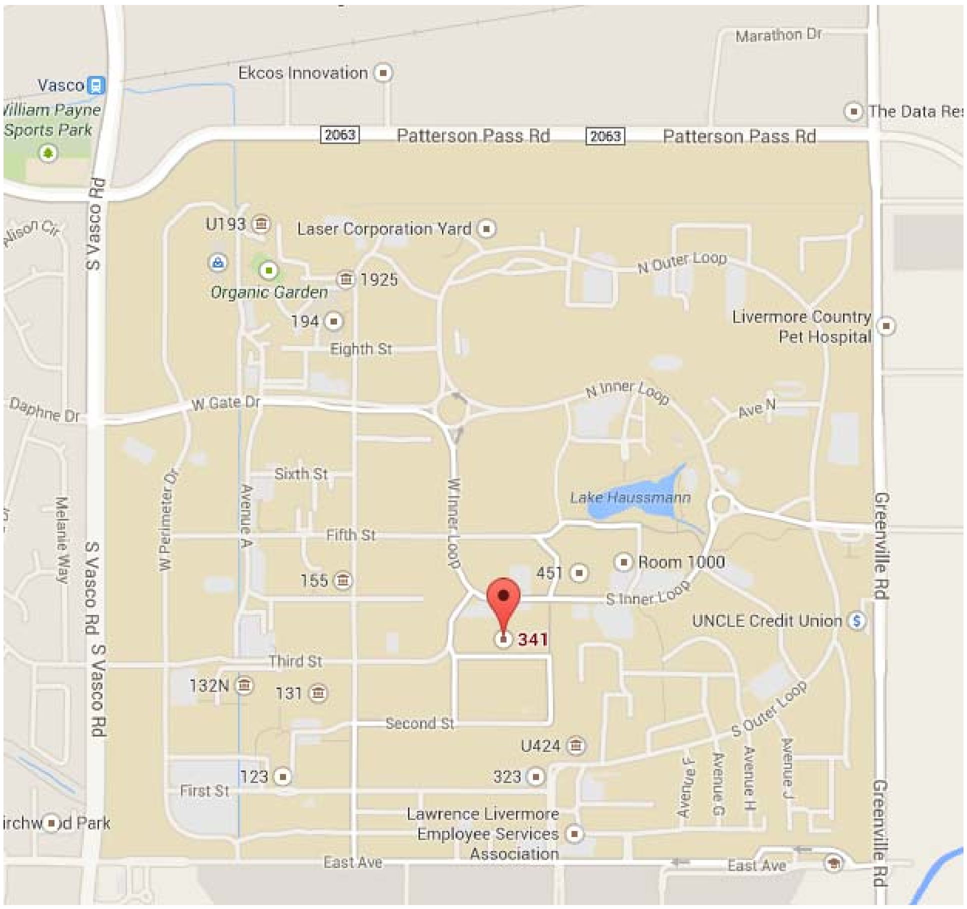


Figure 1 – Lawrence Livermore National Laboratory Vicinity Map

Several soil boring logs were included on the Increment 1 drawings. The soil boring logs indicate fairly stiff soil, with blow counts of 20 to 30 blows/foot over the top 12 feet and greater than 50 blows/foot below a depth of 12 feet. Based on the soil boring data we have assumed a site class D in accordance with ASCE 41-13.

In accordance with the procedures in ASCE 41-13 and after consultation with the Lab, we have used the BSE-1E earthquake as the seismic hazard level for our evaluation of the structures. The BSE-1E corresponds to a uniform hazard spectrum with a 20% probability of exceedence in 50 years. We obtained seismic hazard values from the United States Geological Survey's (USGS) online web application and modified them for Site Class D. The following values were used to construct the General Response spectrum in accordance with ASCE 41-13:

- S_{XS} = 0.98 g
- S_{X1} = 0.52 g

As a point of comparison DOE Standard 1020-2002 references the 2000 International Building Code (IBC) for seismic hazard level for PC-1 and DOE Standard 1020-2012 references ASCE 7-10 for seismic hazard level for PC-1. Based on data from USGS, the following seismic design values were obtained for IBC 2000 and ASCE 7-10, both of which are based on a hazard equal to two-thirds of the Maximum Considered Earthquake:

1. IBC 2000
   a. S_{DS} = 1.11 g
   b. S_{1S} = 0.62 g
2. ASCE 7-10
   a. S_{DS} = 1.33g
   b. S_{D1} = 0.76 g

Figure 2 shows a comparison of all three response spectrum derived from the seismic hazard values.

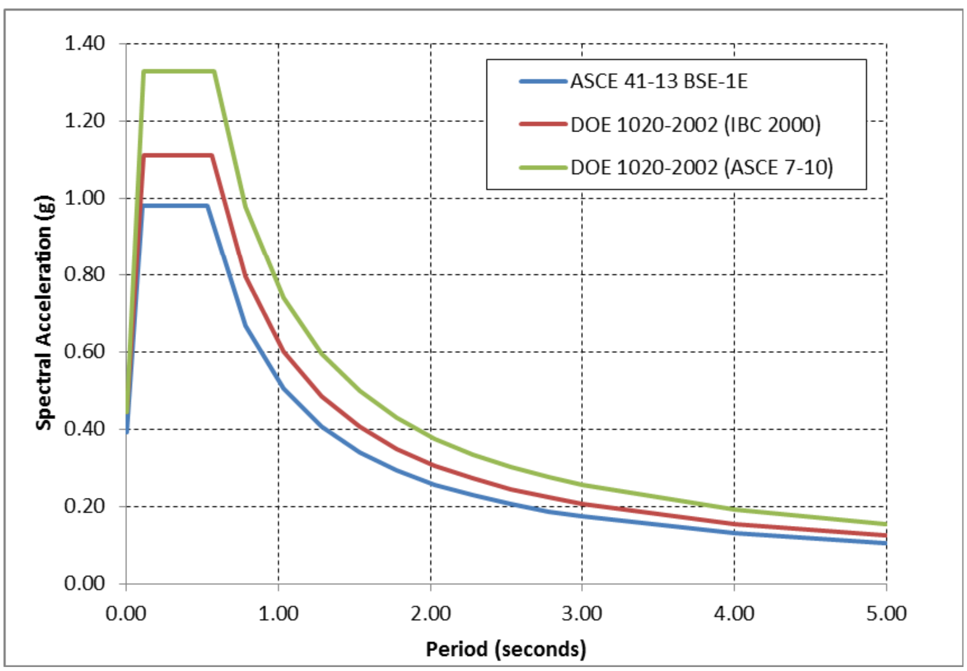


Figure 2 – Comparison of Response Spectrum

3.0 Building Description

Building 341, originally named the Pulsed Energy Research Building, was constructed in three phases. The main building, Increment 1, was constructed in approximately 1963. Two additions were later made to the building; Increment 2, in approximately 1973 and Increment 3 in approximately 1975. Increment 1 is a one-story structure with high roof and low roof areas. There is a partial mezzanine in one half of the high roof area. Increment 2, which is an independent structure, is one-story in height and is located on the west side of Increment 1. Increment 3 is a one-story structure located on the North side of Increment 1. Increment 3 has its own lateral force resisting system, but is structurally connected to the North wall of Increment 1.

A key plan of the building, which identifies all three increments, is shown in Figure 3.

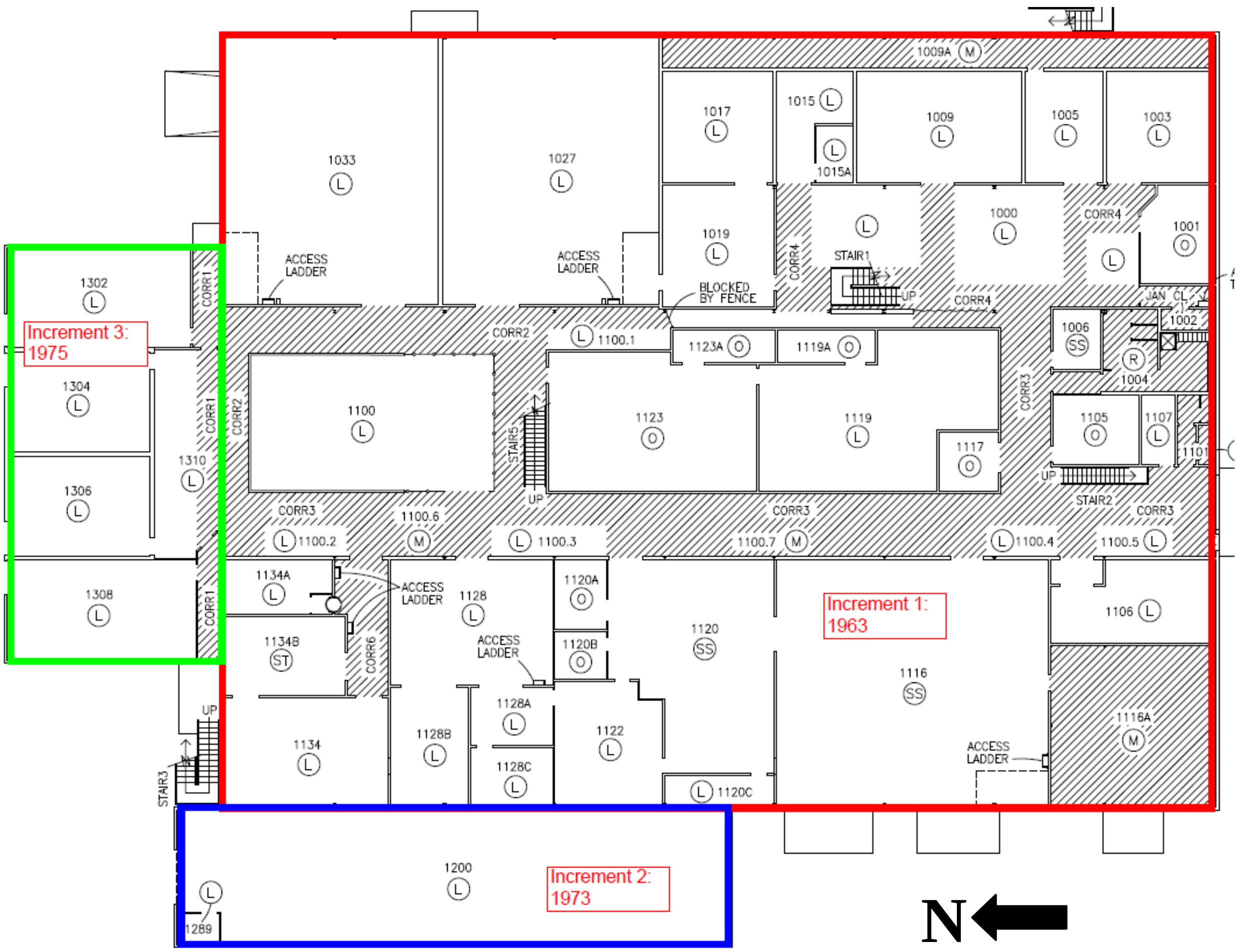


Figure 3 – Building 341 Key Plan

3.1 Building Structure

3.1.1 Increment 1

Increment 1 is a one-story steel and concrete building with a partial mezzanine and exterior precast concrete tilt-up panels. The building is rectangular in plan with overall dimensions of 180 feet in the North-South direction and 140 feet in the East-West direction. A 3D image of the building structure is shown in Figure 4.

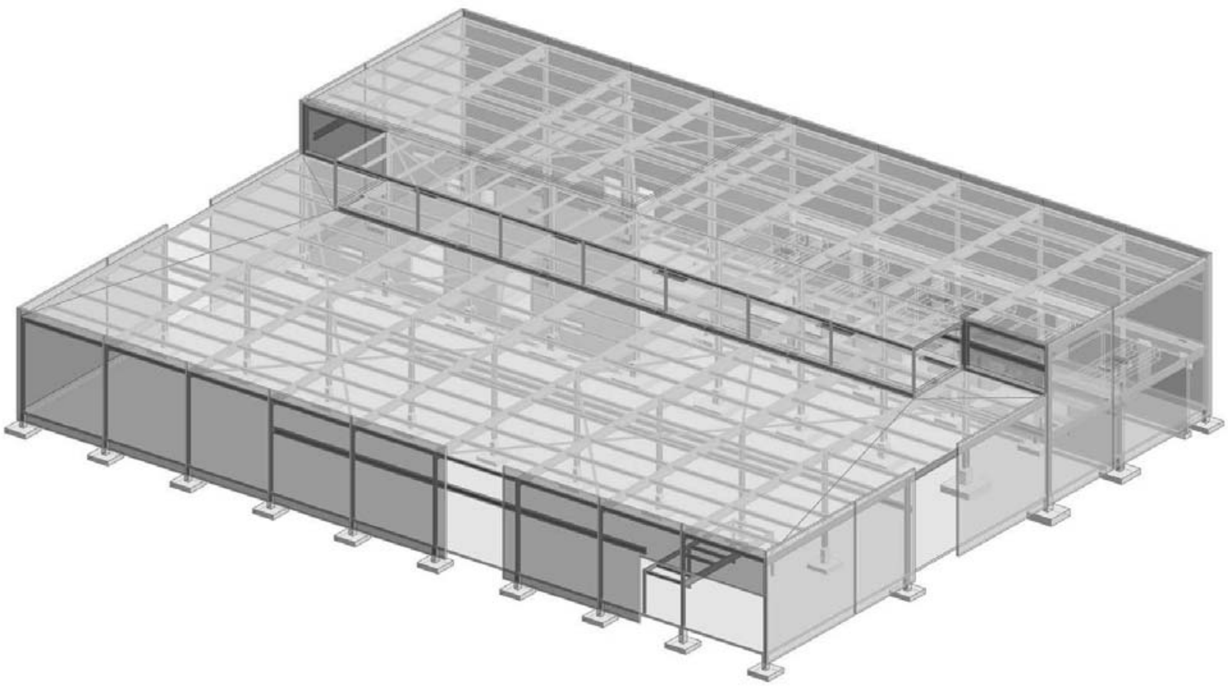


Figure 4 – 3D Image of Increment 1

The original foundation plan is shown in Figure 5.

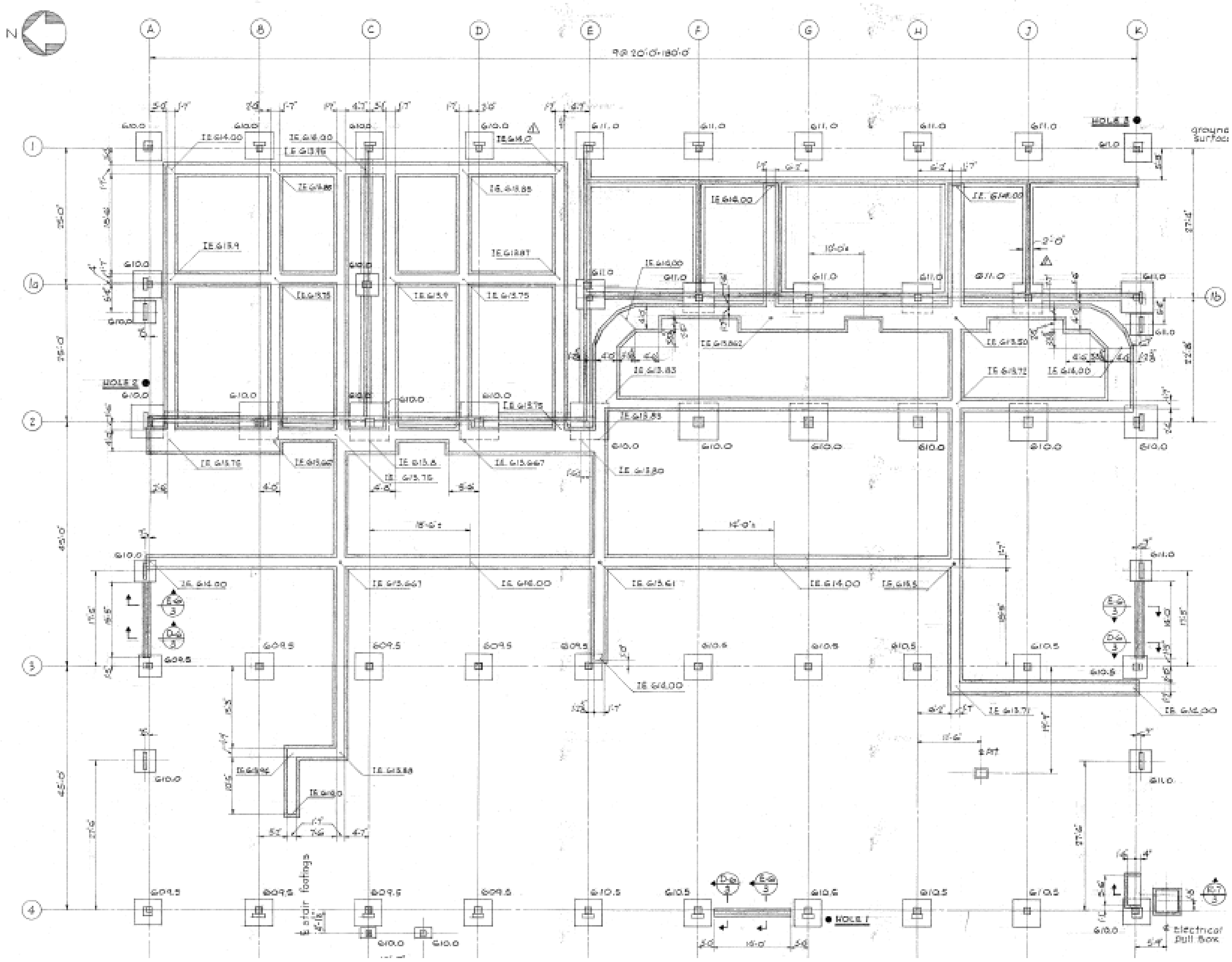


Figure 5 – Increment 1 Foundation Plan

The building consists of a high roof and low roof area. Building height at the high roof areas is approximately 32 feet and building height at the low roof areas is approximately 22 feet. The high roof area has a width of approximately 67 feet in the East-West direction, a portion of which extends over a mechanical room located on the low roof. Figure 6 shows an East-West cross-section through the building, where the step in roof elevation can be seen.

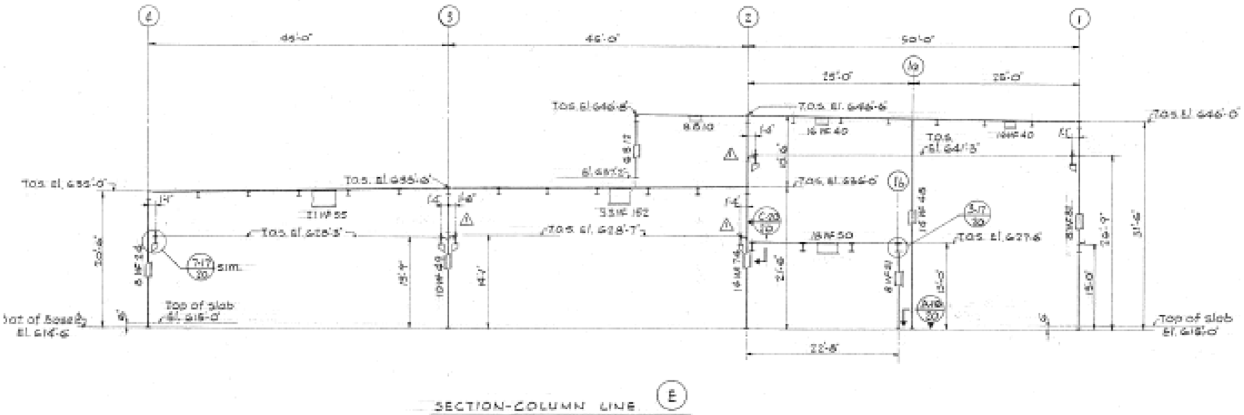


Figure 6 – Increment 1 East-West Cross-Section

There is a partial mezzanine located in the southern half of the high roof area, between grid lines E to K and 1 to 2. The mezzanine has plan dimensions of the 100 feet in the North-South direction and 50 feet in the East-West direction and a finished floor elevation of 13 feet.

At the low roof there is a mechanical room, which extends between grid lines B to J and 2 to 2a. The mechanical room has plan dimensions of approximately 140 feet in the North-South direction and 17 feet in the East-West direction. The southwest corner of the mechanical roof is shown in Figure 7.

Figure 7 – Southwest Corner of Mechanical Room

Gravity Load System

Increment 1's main gravity load resisting system consists of 18 gauge metal deck, supported by wide flange steel beams, girders and columns. At the mezzanine, gravity loads are supported by a reinforced concrete slab, which spans between either steel wide flange beams, girders and columns or reinforced concrete beams and walls. At the mechanical room, the gravity load resisting system consists of a reinforced concrete slab, which spans between steel wide flange beams, girders and columns. Foundations consist of spread footings beneath columns and strip footings beneath cast-in-place concrete walls. The precast concrete panels span between the building columns and are supported by the spread footings located at building columns.

Lateral Force Resisting System

The lateral force resisting system consists of both precast and cast-in-place reinforced concrete shear walls. The precast walls are located at the exterior of the building. The cast-in-place walls are located in the high roof area on lines C, E and 2 and below the mezzanine in both the East-West and North-South directions. The precast walls are six inches thick and the cast-in-place walls are 8 inches thick.

At the exterior of the building and at the cast-in-place concrete walls at the high roof area, lateral loads are delivered to the shear walls by an 18 gauge metal deck, which is puddle welded to the supporting steel beams. In addition to the metal deck diaphragm, horizontal double angle bracing was also provided in the plane of the low and high roof levels, which also deliver loads to the shear walls. The double angle bracing at the low roof level can be seen in Figure 8. At the mezzanine and mechanical room, lateral loads are delivered to the shear walls by the reinforced concrete slab. Horizontal shear loads from the precast panels are delivered to the soil by dowels into the building slab-on-grade and overturning forces are delivered to the building columns and their supporting spread footings by welded concrete inserts between the panels and columns.

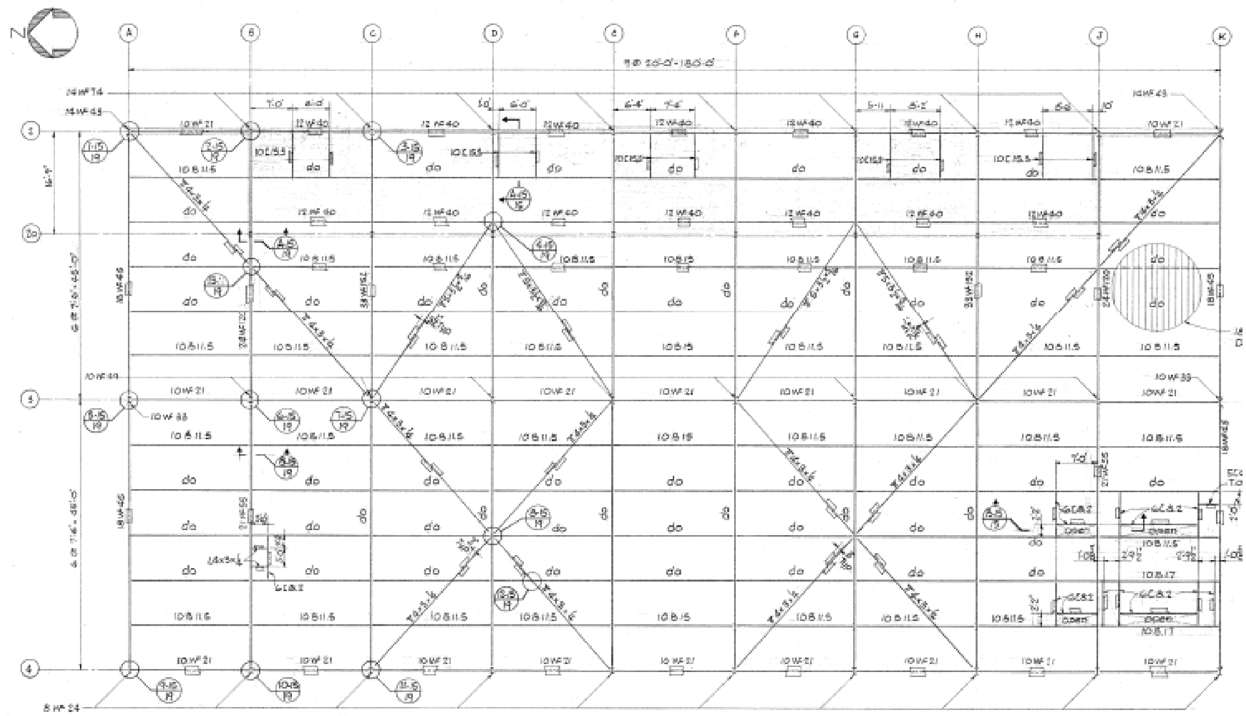


Figure 8 – Increment 1 Low Roof Framing Plan

3.1.2 Increment 2

Increment 2 is a one-story steel framed building with exterior precast concrete tilt-up panels. Increment 2 is located on the West elevation of Increment 1 and the east wall of Increment 2 is created by the West wall of Increment 1. The building is rectangular in plan with overall dimensions of 102 feet in the North-South direction and 25 feet in the East-West direction. Building height is approximately 20 feet. The original roof framing plan is shown in Figure 9.

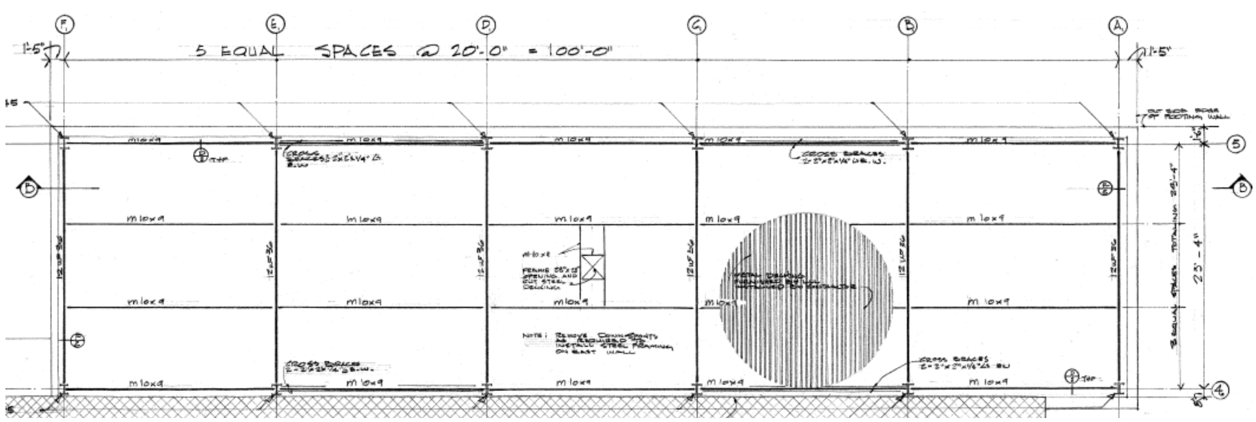


Figure 9 – Increment 2 Roof Framing Plan

Gravity Load System

Increment 2's gravity load resisting system consists of metal deck, supported by wide flange steel beams, girders and columns. Columns are supported by spread footings along the west side of the building and by a strip footing adjacent to Increment 1.

Lateral Force Resisting System

Lateral loads are resisted by several different systems. In the East-West and North-South directions the exterior precast panels are directly attached to the roof diaphragm and will therefore participate in resisting lateral loads. In addition to the precast panels, in the East-West direction it appears that there is a steel moment frame on every column line and in the North-South direction there are two tension only steel braced frames on each line as shown in Figure 10. Lateral loads are delivered to the vertical lateral force resting elements by the metal deck diaphragm.

Figure 10 – Increment 2 Tension Only Bracing

3.1.3 Increment 3

Increment 3 is a one-story concrete and wood framed structure. Increment 3 is located on the North elevation of Increment 1 and the south wall of Increment 3 is created by the North wall of Increment 1. Furthermore, a portion of the Increment 3 roof is supported directly by the precast panels on the North elevation of Increment 1.

The building is rectangular in plan with overall dimensions of 39 feet in the North-South direction and 76 feet in the East-West direction. Building height is approximately 12 feet. The original roof framing plan is shown in Figure 11.

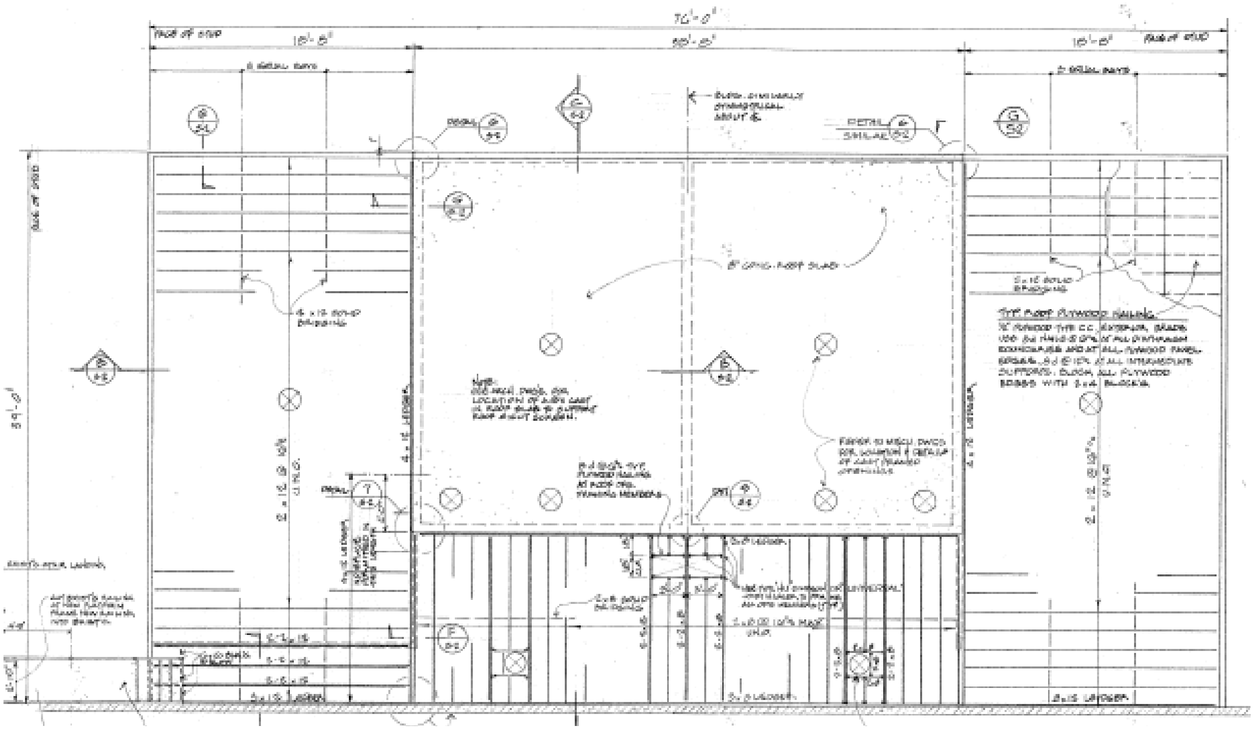


Figure 11 – Increment 3 Roof Framing Plan

Gravity Load System

Increment 3 consists of a concrete core, where gravity loads are supported by a reinforced concrete slab and walls. Around three sides of the concrete core is a wood framed structure, where loads are gravity loads are supported by wood joists, which span between the concrete core and exterior wood framed stud walls, except on the south side, where the joists span to the exterior wall of Increment 1. The concrete and wood stud walls are supported on continuous strip footings.

Lateral Force Resisting System

Lateral loads are resisted by reinforced concrete walls located at the central concrete core structure. Lateral loads are delivered to the concrete walls by a reinforced concrete slab as well as a plywood panel diaphragm located on three sides of the concrete core.

3.2 Condition Assessment

The buildings were constructed between approximately 1963 and 1975 with what appears to be a typical level of construction quality for the time period. During our site visit we did not observe any significant signs of deterioration. We did observe a small amount of concrete spalling at several of the precast panel connections on the east wall of Increment 1. In general, the buildings appear to be in good physical condition and have been well maintained.

4.0 Seismic Evaluations of Increments 2 and 3

As described in our proposal we performed Tier 1 Seismic Evaluations of Increments 2 and 3 in accordance with ASCE 41-13. Tier 1 Evaluations consist of completing a series of checklists and simple “quick check” calculations of the lateral force resisting system. In agreement with PC-1 the selected seismic performance level for the evaluations was Life Safety (S-3) in accordance with ASCE 41-13.

We have assumed, that where cranes are present within the buildings, that they will be parked at the end of the rails and that catches are provided to ensure that the cranes will remain properly seated on the rails during a seismic event.

4.1 Increment 2

Given that there are three systems that contribute to resisting lateral forces in Increment 2, in addition to the Life Safety Basic Configuration Checklist, checklists for the following systems were completed:

- Steel Moment Frames with Stiff or Flexible Diaphragms (S1/S1A)
- Steel Braced Frames with Stiff or Flexible Diaphragms (S2/S2A)
- Precast/Tilt-up Concrete Shear Walls Stiff or Flexible Diaphragms (PC1/PC1A)

Completed checklists as well as accompanying structural calculations are contained in Appendix B.

Based on the checklists and associated calculations, the following deficiencies were identified:

- Adjacent Buildings. There is only a 2-inch gap between the columns of Increment 2 and the exterior wall of Increment 1. It is likely that pounding will occur at this location, causing damage to the wall of Increment 1 and to the steel framing of Increment 2. Given the relatively low weight of Increment 2 and the relatively long length over which contact will occur, we do not believe the pounding damage will lead to a collapse, and therefore is not a life-safety concern.
- Torsion. Due to the long line of precast panels on the west side of the building and the lack of panels on the east side of the building there is a large eccentricity between the center of mass and the center of rigidity. However, given that the roof is a flexible diaphragm, this is not a life-safety concern.
- Moment Frame Beam-Column Connections. The drawings do not provide details of the beam-column moment connections. However, based on visual observation of the connections and an evaluation of the connections demands based on the capacity of the panel zones it appears that there is likely adequate strength. The system is also expected to undergo limited inelastic action, therefore we do not believe this is a life-safety concern.

- Moment Frame Panel Zones. Column panel zones do not have the shear capacity to resist the shear demand required to develop 80% of the flexural strength of the beams. However, given the low moment frame bending demand-capacity ratio from the quick check and the presence of the diaphragm and concrete shear walls located at the ends of the building, we do not believe this is a life-safety concern. Furthermore, a quick Tier 2 check reveals that the panel zones have adequate strength when evaluated for the actual shear demand and the appropriate m-factor.
- Moment Frame Drift Check. The lateral drift of the moment frames calculated using the quick check procedure and a 2-dimensional ETABS model is 5.4%, which exceeds the acceptable limit of 2.5%. Other than local damage associated with pounding at the exterior wall of Increment 1, we do not believe this is a life-safety concern.
- Transfer to Steel Frames. The original construction documents do not indicate the type and frequency of attachment between the metal deck and the moment frames. However, given that every transverse column line is a moment frame and the concrete panels are directly connected to the columns and beams, there is very little demand on the diaphragm, therefore, we do not believe this is a life-safety concern. In addition, there is probably some nominal connection between the deck and beams.
- Moment Frame Redundancy. The number of bays per moment frame line is less than 2. However, given that every transverse line is a moment frame the building has significant redundancy, therefore we do not believe this is a life-safety concern.
- Moment Frame – Interfering Walls. The precast panels on the North and South sides of the building are in the same plane as the moment frames and will resist lateral loads. Per the precast panel quick check, the panels have adequate capacity to resist the lateral loads they are subjected to, therefore, this is not a life-safety concern.
- Braced Frame Connection Strength. The brace connections do not develop the yield capacity of the diagonal braces.
- Braced Frame Joints. The diagonal braces do not frame into the beam-column joints concentrically. There is an eccentricity in the connections at each end of the braces.

4.2 Increment 3

In addition to the Life Safety Basic Configuration Checklist the checklist for Concrete Shear Walls with Stiff or Flexible Diaphragms (C2/C2A) was completed.

Completed checklists as well as accompanying structural calculations are contained in Appendix C.

Based on the checklists and associated calculations, the following deficiency was identified:

- Adjacent Buildings. The south side of the wood framed roof of Increment 3 is directly supported by the exterior precast concrete panels on the North elevation of Increment 1. This could lead to some damage to the roof structure of Increment 3 due to differential movement between the two structures. We do not believe this is a life-safety concern given the relatively stiff lateral force resisting systems present in both structures.

5.0 Seismic Evaluation of Increment 1

Increment 1 was evaluated in accordance with ASCE 41-13 using a Tier 3 procedure. A Tier 3 evaluation consists of a detailed analysis of the building structure using a 3-dimensional model and involves a complete check of the lateral load resisting system and its elements. In agreement with PC-1 the selected seismic performance level for the evaluation was Life Safety (S-3) in accordance with ASCE 41-13.

Detailed structural calculations that were performed for the Tier 3 evaluation are included in Appendix D.

5.1 Summary of Tier 3 Seismic Evaluation

The building was analyzed and evaluated in accordance with ASCE 41-13 Tier 3 using a linear dynamic procedure. A three-dimensional model of the building was created in SAP2000. A 3D perspective view of the model is shown in Figure 12. The concrete shear walls, concrete slabs and steel roof diaphragm were explicitly modeled using shell elements. The double angle bracing and beams and columns, where needed, were modeled using beam-column elements. The stiffness of the shell elements was modified to account for cracking based on the recommendations in ASCE 41. Material properties were based upon the structural drawings as well as the preliminary evaluation document contained in Appendix A.

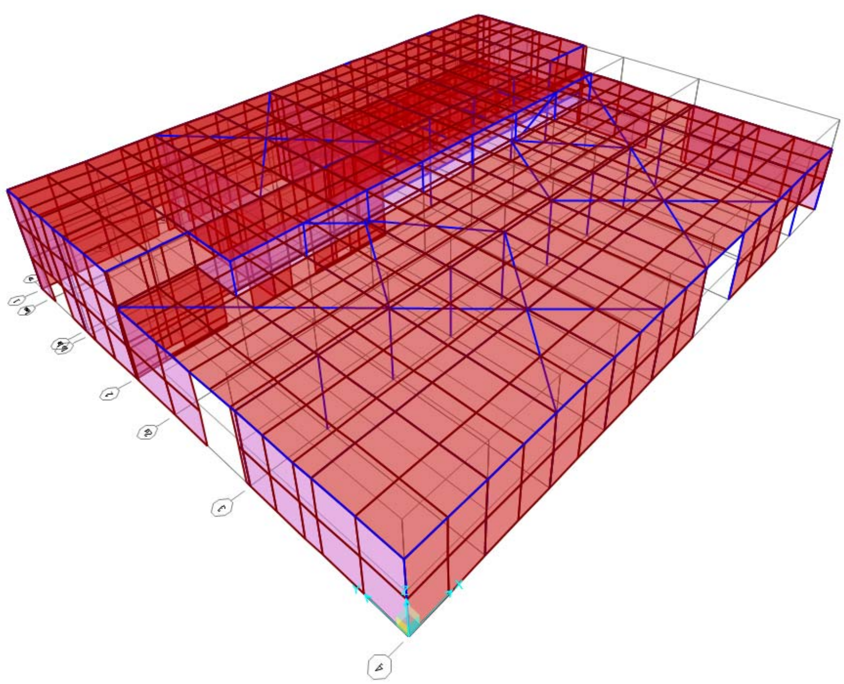


Figure 12 – 3D Perspective View of Analysis Model

Self-weight of the concrete walls and slabs was directly used for assembling the model mass. A distributed area mass was applied to the roof diaphragms based on a separate weight takeoff. The model was analyzed using the General Response Spectrum developed for the BSE-1E earthquake following the procedure in Section 2.4.1.7 of ASCE 41. The General Response Spectrum is shown in Figure 13.

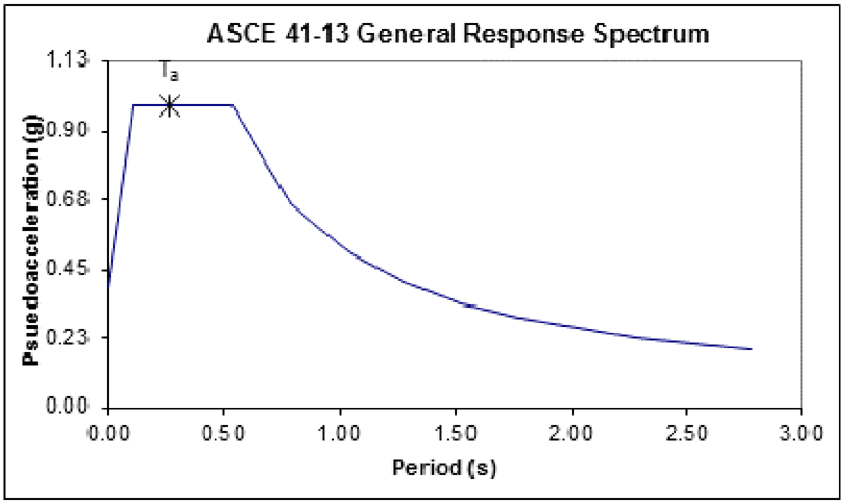


Figure 13 – ASCE 41-13 General Response Spectrum

Modal damping of 5% was used in the analysis. Forty modes were used in the analysis to achieve a total of 95% mass participation in each of the principal horizontal axes.

Based on the analysis model, the fundamental period of the fixed based structure is approximately 0.29 seconds in the transverse (East-West) direction and 0.21 seconds in the longitudinal (North-South) direction.

Per the requirements of ASCE 41-13, the demands from the model on deformation controlled (ductile) elements were further increased by the product of the factors C_1 and C_2, which was taken from Table 7-3, and is equal to 1.4. This was accomplished by scaling the general response spectrum up by this value. The model demands on deformation controlled elements were then reduced by the appropriate m-factor per ASCE 41-13. The demands on force controlled (brittle) elements taken from the model were reduced by the product of the factors C_1 and C_2 per ASCE 41-13.

We have assumed, that where cranes are present within the building, that they will be parked at the end of the rails and that catches are provided to ensure that the cranes will remain properly seated on the rails during a seismic event.

5.2 Results of Tier 3 Seismic Evaluation

Based on the Tier 3 seismic evaluation of Increment 1 the building does not meet a Life Safety performance level for the BSE-1E earthquake ground shaking hazard. The evaluation has resulted in the identification of the following deficiencies.

- Precast panel to panel connection along the vertical joint are overstressed as they attempt to transfer shear forces between adjacent panels. The demand on the vertical panel connections results from the panels attempting to act as a single reinforced concrete shear wall. The connections are overstressed on Lines 4, A and K.
- The roof diaphragm at the high roof along lines E and K is overstressed in shear.
- There is a lack of a collector connection between the low roof framing and the concrete shear walls on lines C and E. There are significant loads that need to be dragged to these walls, yet there is no direct connection.
- There is a lack of a collector connection between the low roof/mechanical roof framing and the concrete shear wall on line 2. There is adequate connection between the concrete slab at the mechanical roof and the concrete wall; however, there is inadequate connections between the beams south of the shear wall that drag the loads to the wall.
- The horizontal double angle diaphragm bracing connections are overstressed at lines 2/A and 2/K. These angles serve to transfer some of the diaphragm forces to the perimeter precast walls. The angles have adequate axial capacity; however, the connections are inadequate.
- When evaluated as force controlled using lower bound material properties, some of the precast concrete walls have inadequate strength to span out-of-plane to supporting elements, such as steel wide flange strong backs or the roof beam. However, when expected material properties are used, the panels have adequate strength. Therefore we do not believe this item is a life-safety concern.
- The precast panel strong back at grid line A/1a, which is also a building column, is slightly overstressed in bending when subjected to out-of-plane wall loading. However, given the flexural yielding is a ductile mechanism, we do not believe this is a life-safety concern.

6.0 Seismic Strengthening

Based on the results of the Tier 1 and Tier 3 seismic evaluations of Building 341, the building does not meet the Life Safety performance level for the BSE-1E earthquake ground shaking hazard. Several deficiencies were identified that require mitigation in order to achieve a Life Safety performance level. The following is a list of seismic strengthening measures, in order of importance, that we recommend be implemented to achieve a Life Safety performance level:

Increment 1

1. Add collector connections between the existing low roof steel beam and the concrete shear walls at lines 2/C and 2/E as shown in Figure 14, Figure 15 and Figure 16.
2. Strengthen the beam-column connections along line 2 at the low roof to transfer the collector demands to the concrete shear wall on line 2 as shown in Figure 14 and Figure 17.
3. Strengthen the beam-column connections at lines 3/C and 3/E at the low roof to transfer the collector demands to the concrete shear walls on lines C and E as shown in Figure 18.
4. Strengthen precast panel to panel connections along vertical joints at lines 4, A and K as shown in Figure 19 (interior of building) or Figure 20 (exterior of building).
5. Strengthen double angle horizontal bracing connections at grid lines 2/A and 2/K by adding welds to the existing connections as shown in Figure 21.
6. Strengthen the high roof diaphragm between lines E and K by adding new horizontal bracing as shown in Figure 22.

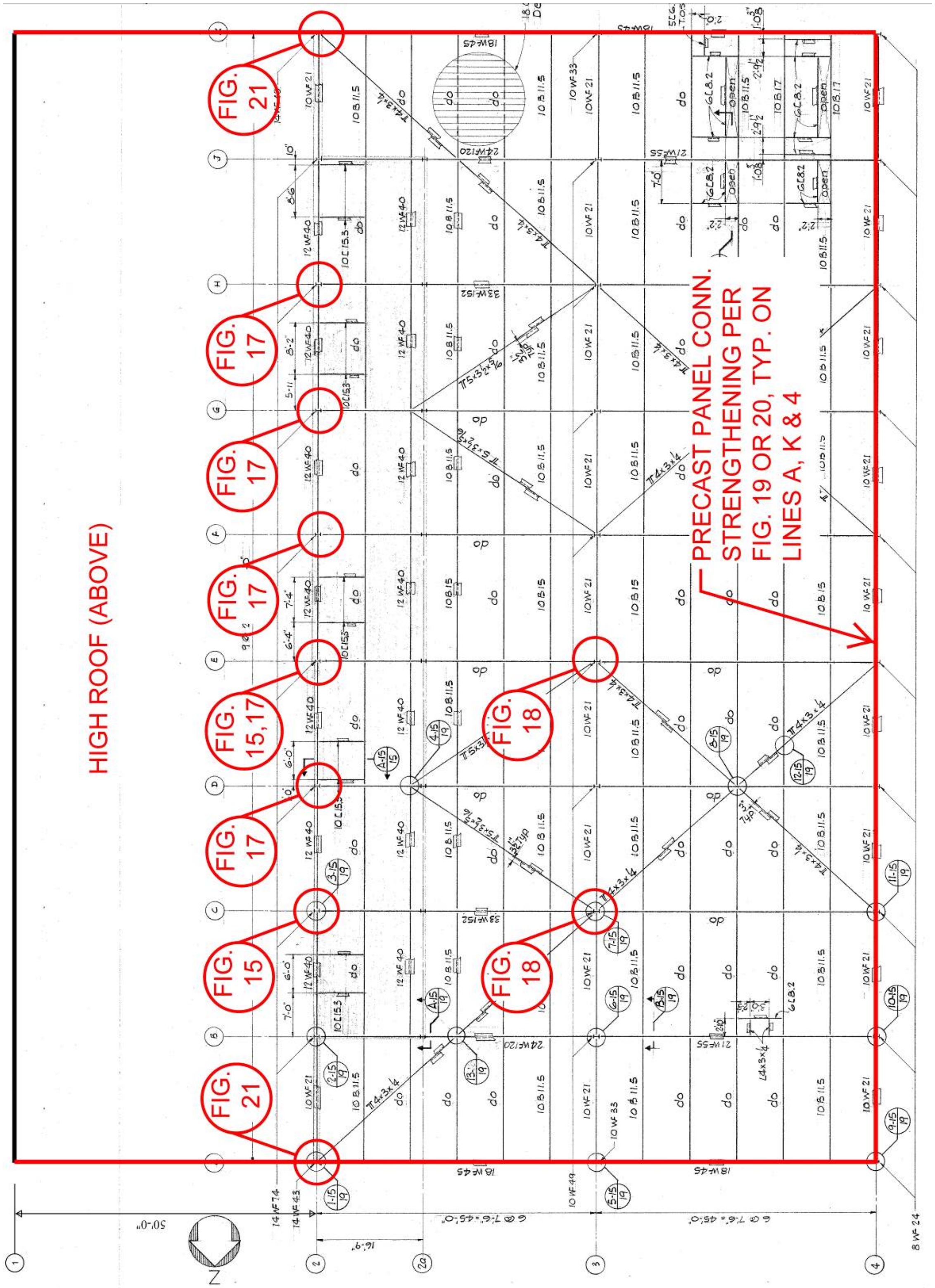


Figure 14 – Low Roof Framing Plan with Seismic Strengthening

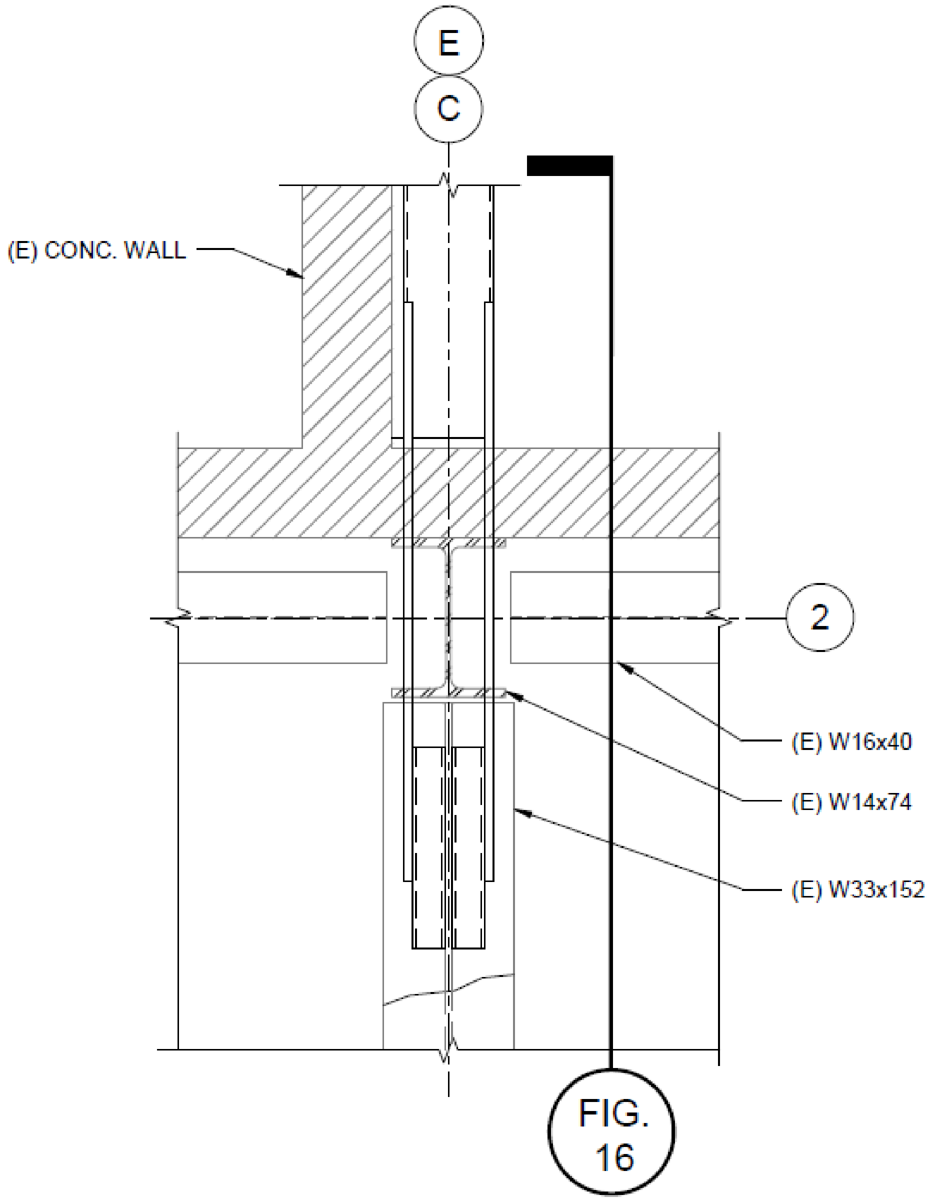


Figure 15 – Collector Connection at Lines 2/C and 2/E

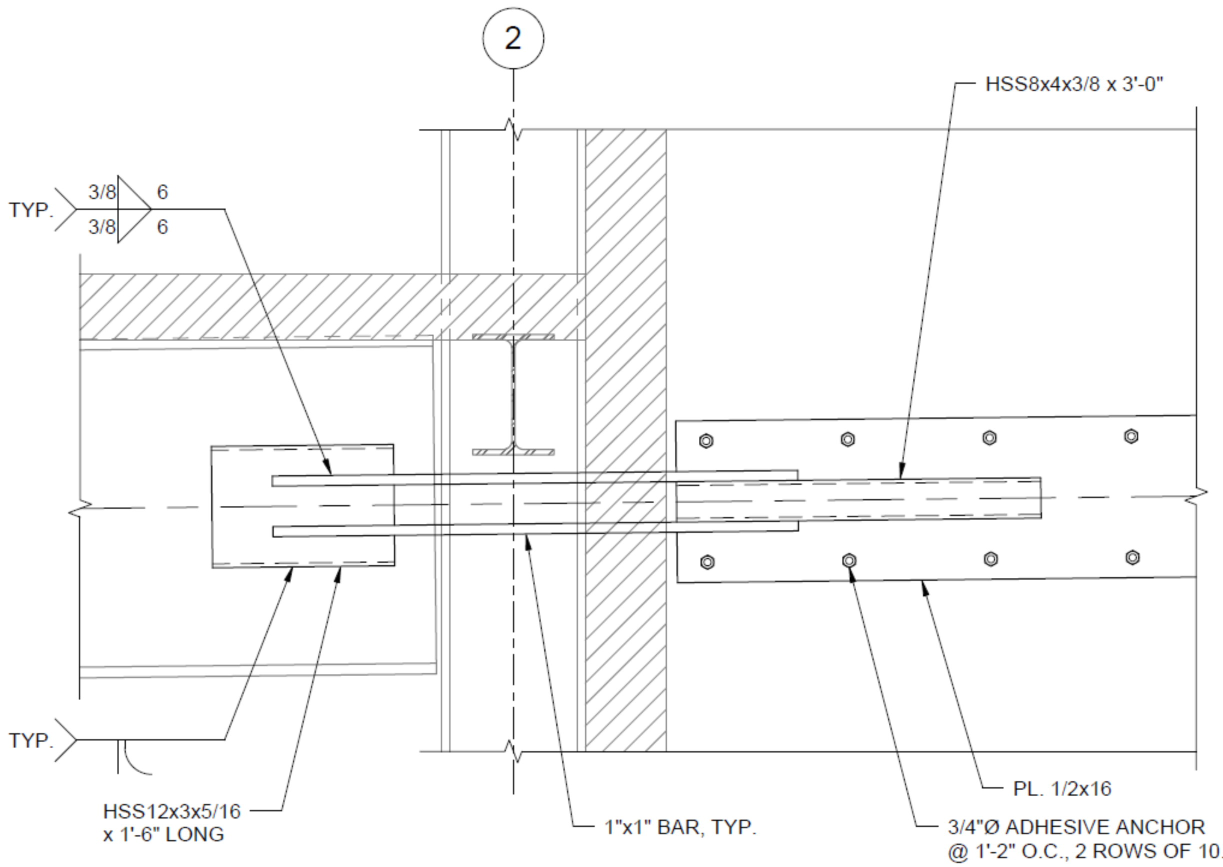


Figure 16 – Collector Connection to Shear Wall at Lines 2/C and 2/E

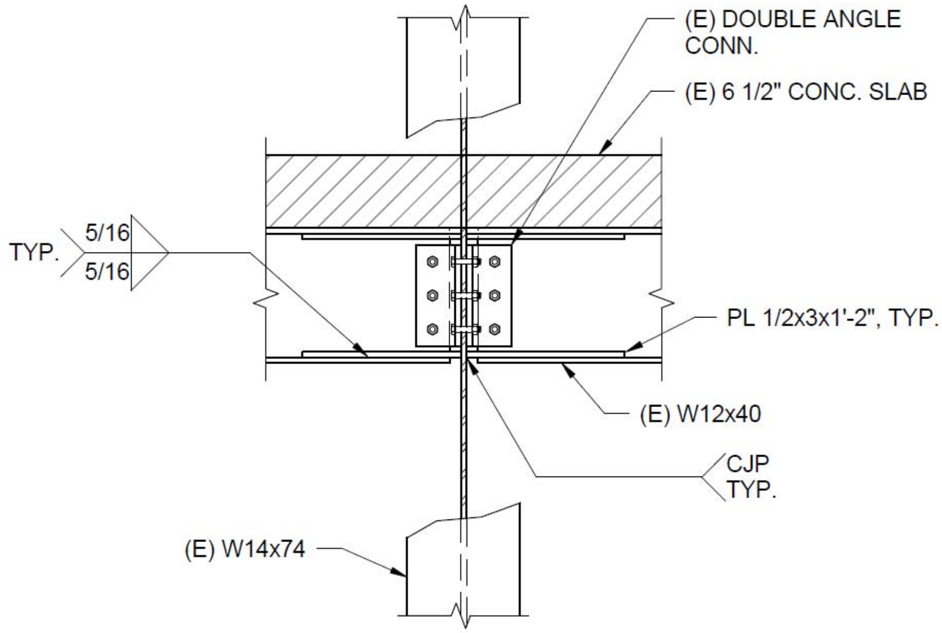


Figure 17 - Collector Connection along Line 2

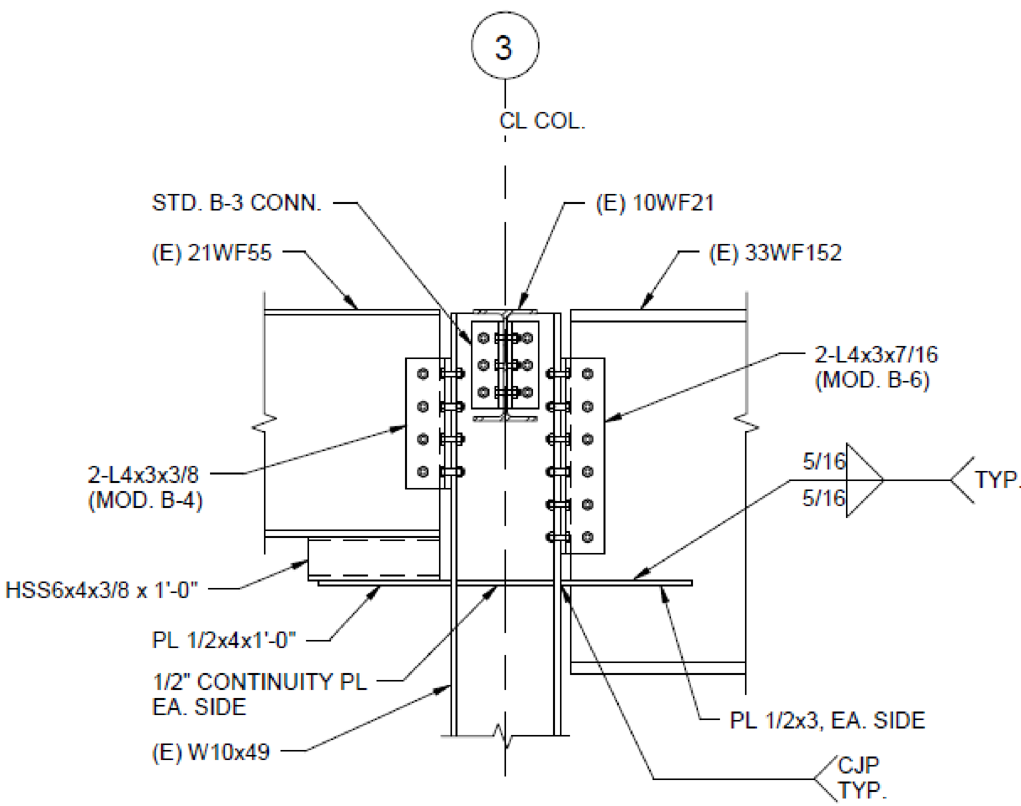


Figure 18 – Collector Connection at Lines 3/C and 3/E

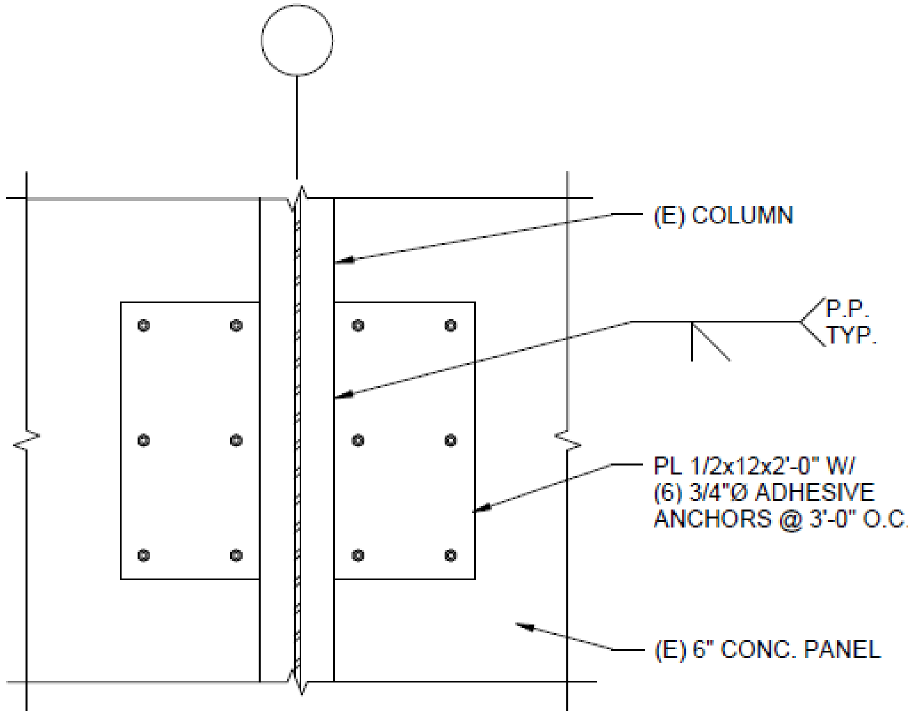


Figure 19 – Precast Panel-Panel Connection Strengthening (Interior Face)

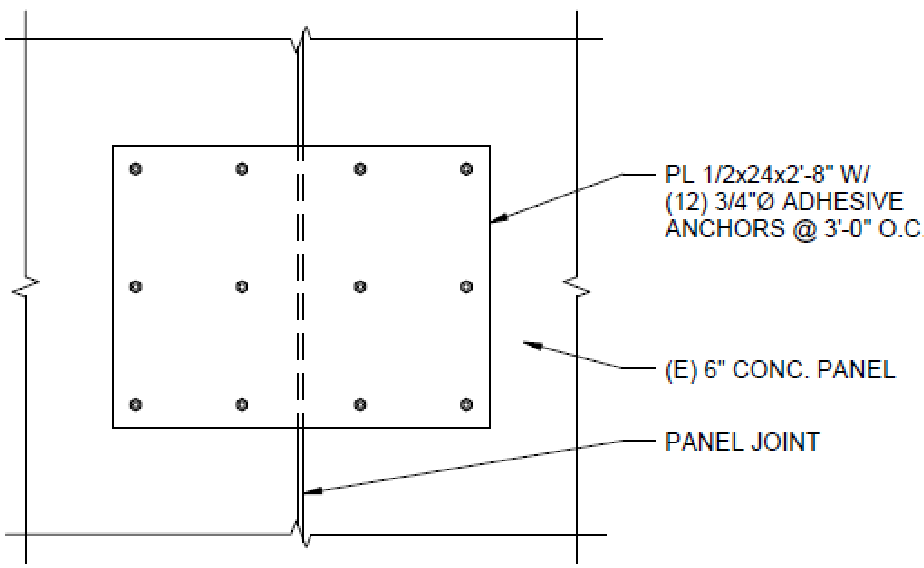


Figure 20 – Precast Panel-Panel Connection Strengthening (Exterior Face)

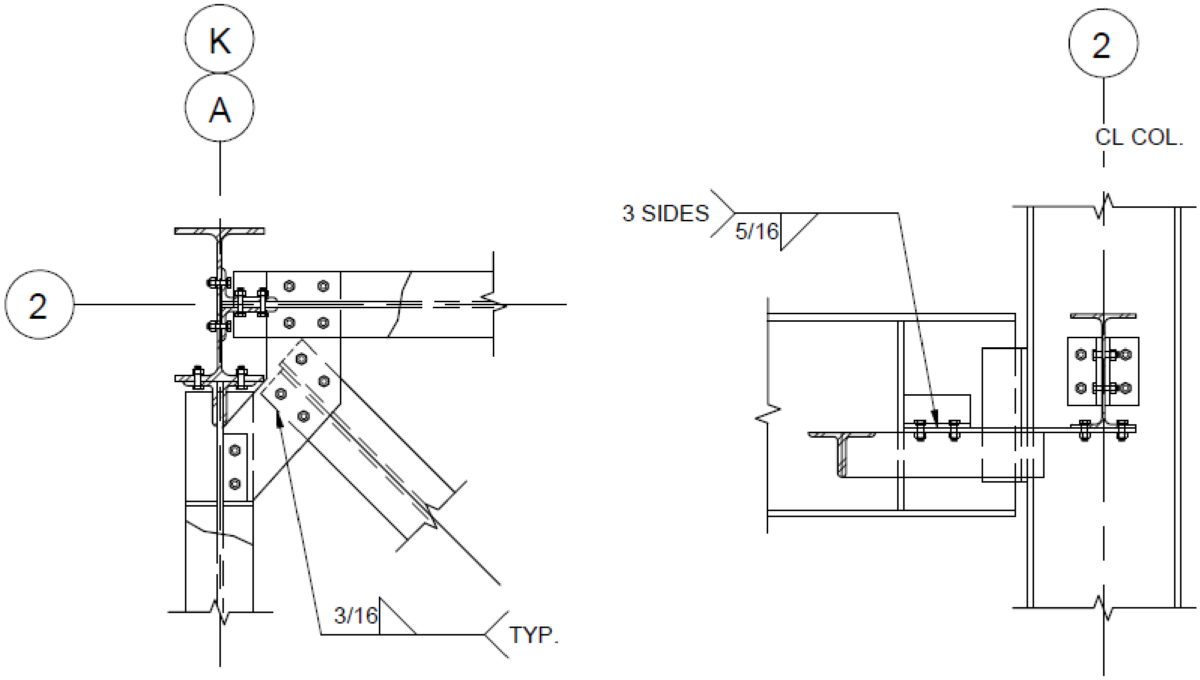


Figure 21 – Strengthening of Existing Horizontal Bracing at Lines 2/A and 2/K

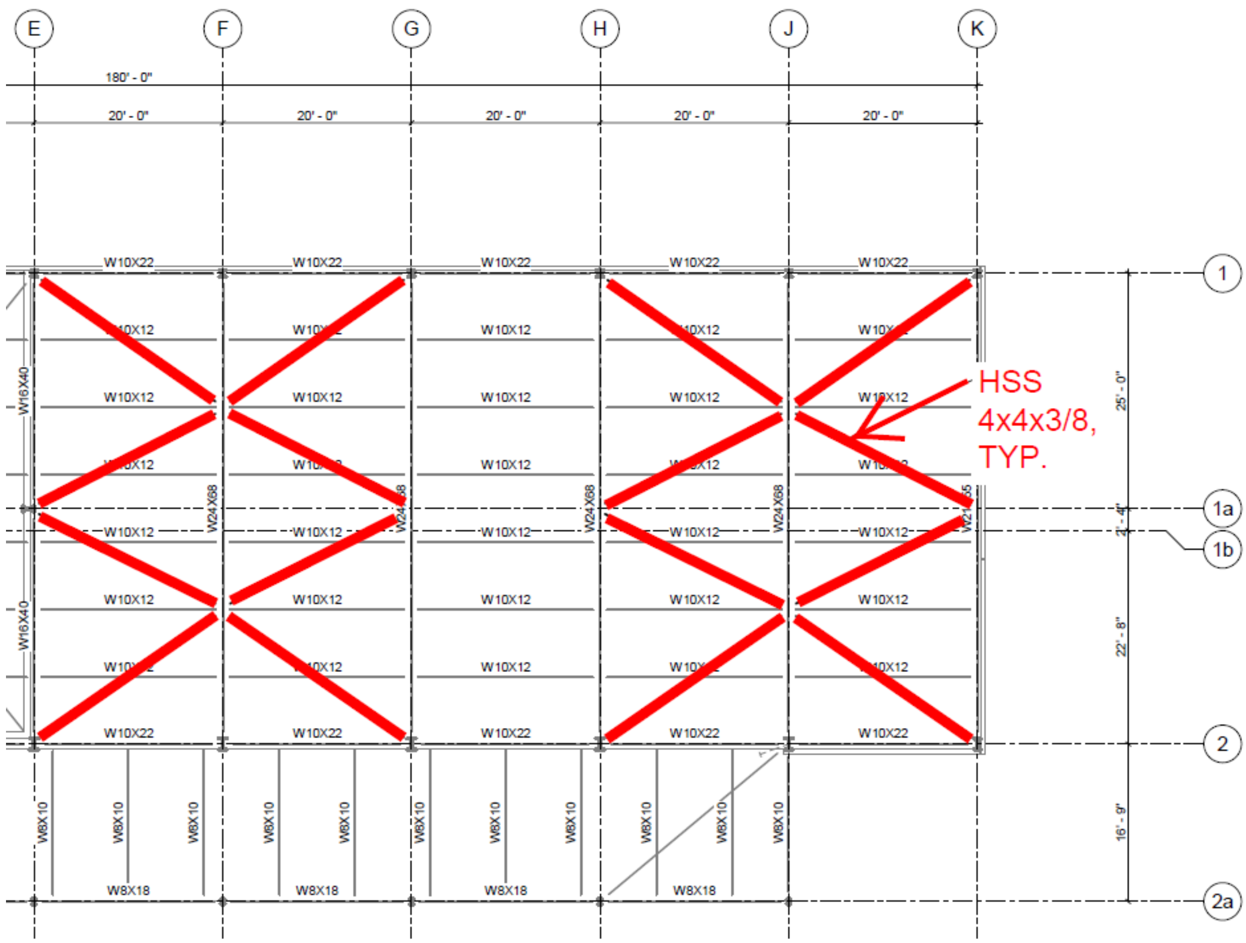


Figure 22 – Partial High Roof Framing Plan - Horizontal Bracing

Increment 2

1. Strengthen the brace connections on the east and west sides of the building to develop the capacity of the diagonal braces. This involves removing the existing gusset plates and replacing with new gusset plates and at the column base, extending the base plate. Strengthening is shown in Figure 23, Figure 24 and Figure 25.

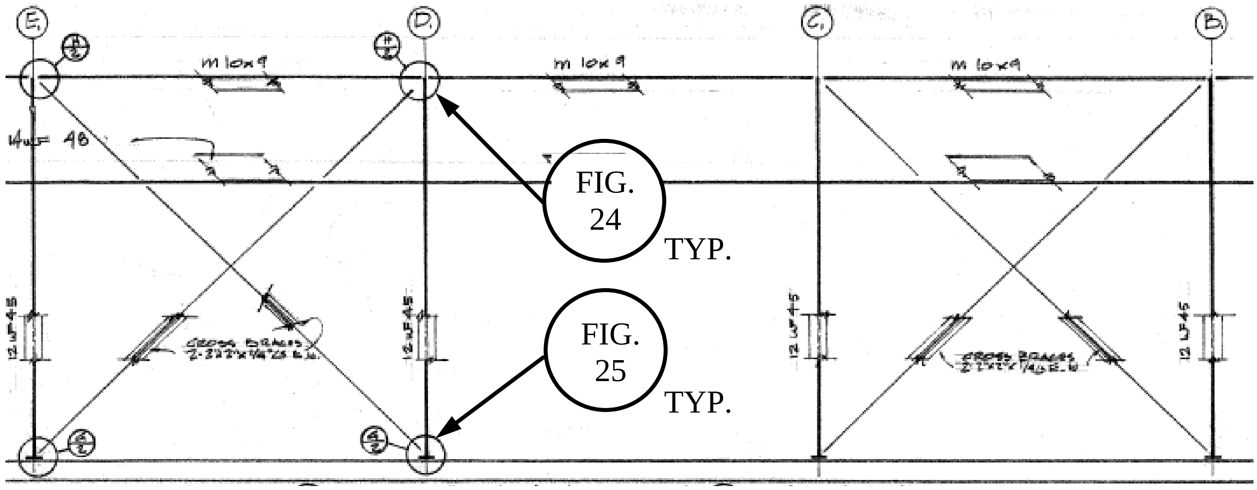


Figure 23 – Increment 2 – East and West Braced Frame Elevations

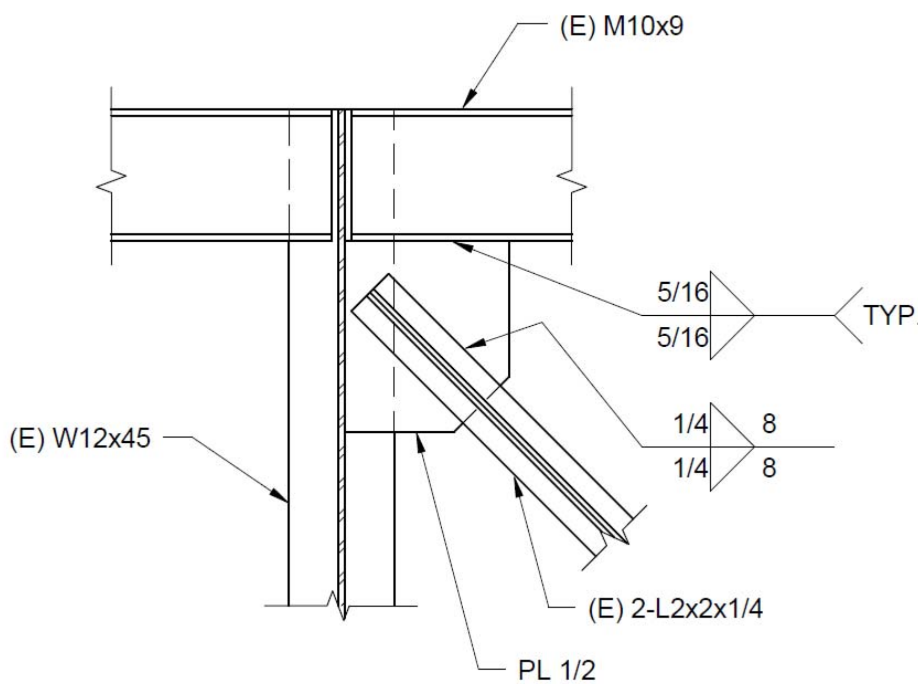


Figure 24 – Strengthening of Tension Only Brace Connection at Roof

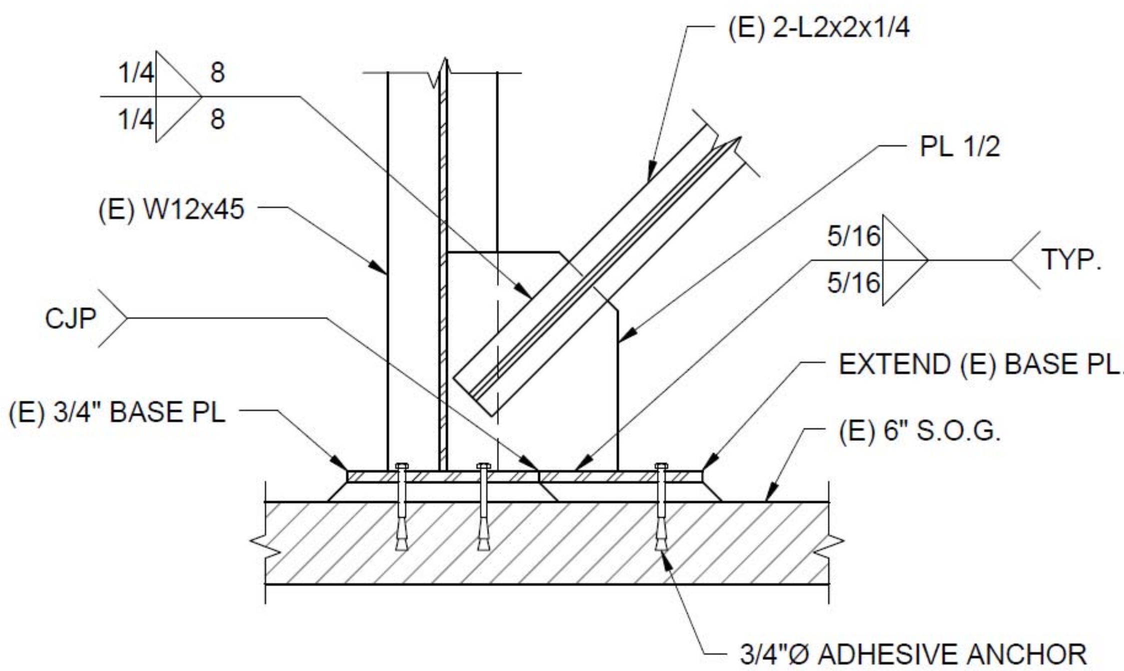


Figure 25 - Strengthening of Tension Only Brace Connection at Column Base

Appendix A: Increment 2 Tier 1 Check Lists and Calculations

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment II | Date: November 13, 2013

Building Address: Lawrence Livermore National Laboratory | Page: 1 of 2

Job Number: B3189012.00 Job Name: LLNL B341 Seismic Evaluation | By: AMN Checked: ______

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**There is only a 2" gap between columns of Increment II and the exterior wall panels of Increment I. There will be limited damage due to pounding, however, we do not believe this is a life-safety concern.**
☐	☐	☒	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☐	☐	☒	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☐	☐	☒	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☐	☒	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**Exterior wall panels on west façade will create torsion in long direction, however, given flexible diaphragm this is not a life safety concern.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	

Building Name: 341, Increment II | Date: November 13, 2013
Building Address: Lawrence Livermore National Laboratory | Page: 2 of 2
Job Number: B3189012.00 | Job Name: LLNL B341 Seismic Evaluation | By: AMN | Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☐	☐	☒	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	**20'/19' = 1.05 > 0.6*0.98 = 0.59**
☐	☐	☒	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	**Foundation tied by slab-on-grade.**

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment II | Date: November 13, 2013
Building Address: Lawrence Livermore National Laboratory | Page: 1 of 3
Job Number: B3189012.00 Job Name: LLNL B341 Seismic Evaluation | By: AMN Checked: ____

ASCE 41-13 S1 / S1A Life Safety Structural Checklist: Steel Moment Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)	
☒	☐	☐	☐	AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)	
☒	☐	☐	☐	FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9 is less than F_y. Columns need not be checked if the Strong Column/Weak beam checklist item is Compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.5)	
				Connections	
☐	☐	☐	☒	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.1)	**Given that every column line is a moment frame and the majority of the building weight is in the concrete panels, which are directly connected to the beams and columns, there is very little demand on the diaphragm, therefore, we do not believe this is a life-safety concern.**
☒	☐	☐	☐	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1)	**Given that every column line is a moment frame, there is significant redundancy in the structure, therefore, this is not a life-safety concern.**
☐	☒	☐	☐	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)	**The concrete panels that are in line with the concrete moment frames at the end bays have adequate strength to resist the applied lateral loads, therefore this is not a life-safety issue.**
☐	☐	☐	☒	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note more restrictive requirements for High Seismicity.	**See comment on next page.**

ASCE 41-13 S1 / S1A Life Safety Structural Checklist: Steel Moment Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Seismic-Force-Resisting System	
☐	☐	☐	☒	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110 percent of the expected yield stress of the steel per AISC 341 Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)	**The actual flexural demand on the beam-column connections based on the column panel zone capacity is approximately 60% of the beam capacity. Based on visual observation of the connections it appears there is adequate strength to develop the panel zone capacity.**
☐	☒	☐	☐	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)	**Using the Tier 3 m-factor and the actual shear demand, the panel zone has adequate strength.**
☐	☐	☒	☐	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)	
☒	☐	☐	☐	STRONG COLUMN / WEAK BEAM: The percentage of strong column / weak beam joints in each story of each line of moment frames is greater than 50 percent. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)	
☒	☐	☐	☐	COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1 for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)	
				Diaphragms (Stiff or Flexible)	
☐	☐	☒	☐	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25 percent of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment II — Date: November 13, 2013

Building Address: Lawrence Livermore National Laboratory — Page: 1 of 2

Job Number: B3189012.00 — Job Name: LLNL B341 Seismic Evaluation — By: AMN — Checked: ______

ASCE 41-13 S2 / S2A Life Safety Structural Checklist: Steel Braced Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				LOW SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces is less than 0.10*Fy*. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than 0.30*Fy*. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.4.1)	
☒	☐	☐	☐	AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than 0.50*Fy*. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)	
				Connections	
☐	☐	☐	☒	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)	**Given the relatively low demand on the diaphragm in the long direction of the building we do not believe this is a life-safety concern.**
☒	☐	☐	☐	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1)	
☐	☐	☒	☐	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)	
☐	☐	☒	☐	COMPACT MEMBERS: All brace elements meet compact section requirements set forth by AISC 360 Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)	
☒	☐	☐	☐	K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	COLUMN SPLICES: All column splice details located in braced frames develop 50 percent of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2)	
☐	☐	☒	☐	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl/r* ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3)	

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment II | Date: November 13, 2013

Building Address: Lawrence Livermore National Laboratory |

Job Number: B3189012.00 Job Name: LLNL B341 Seismic Evaluation | By: AMN Checked:

ASCE 41-13 S2 / S2A Life Safety Structural Checklist: Steel Braced Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☐	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)	
☒	☐	☐	☐	COMPACT MEMBERS: All brace elements meet section requirements set forth by AISC 341 Table D1.1 for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)	
☐	☐	☒	☐	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs. (Commentary: Sec. 3.3.2.3. Tier 2: Sec. 5.5.4.6)	
☐	☒	☐	☐	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces shall frame into the beam-column joints concentrically. (Commentary: Sec. 3.3.2.4. Tier 2: 5.5.4.8)	
				Diaphragms (Stiff or Flexible)	
☐	☐	☒	☐	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25 percent of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Degenkolb
DEGENKOLB ENGINEERS

Building Name: **341, Increment II** Date: **November 13, 2013**

Building Address: **Lawrence Livermore National Laboratory** Page: **1** of **2**

Job Number: **B3189012.00** Job Name: **LLNL B341 Seismic Evaluation** By: **AMN** Checked: ______

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C NC N/A U **Comments**

LOW SEISMICITY

Connections

☒ ☐ ☐ ☐ WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)

MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)

Seismic-Force-Resisting System

☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

☒ ☐ ☐ ☐ SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)

☒ ☐ ☐ ☐ REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)

Diaphragms

☐ ☐ ☒ ☐ TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)

Connections

☐ ☐ ☒ ☐ WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)

☒ ☐ ☐ ☐ TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

☐ ☐ ☒ ☐ TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)

☒ ☐ ☐ ☐ GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

☐ ☐ ☒ ☐ DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

☒ ☐ ☐ ☐ WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)

Diaphragms

☒ ☐ ☐ ☐ CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)

☐ ☐ ☒ ☐ STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

☐ ☐ ☒ ☐ SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

☐ ☐ ☒ ☐ UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

☒ ☐ ☐ ☐ OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Connections

☒ ☐ ☐ ☐ MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)

☒ ☐ ☐ ☐ PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)

☐ ☐ ☒ ☐ UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)

☐ ☐ ☒ ☐ GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)

USGS Design Maps Summary Report

User–Specified Input

Report Title LLNL - Building 341
Wed November 13, 2013 18:35:49 UTC

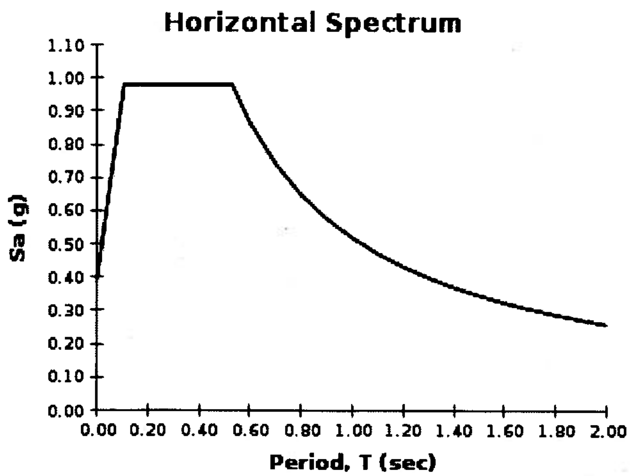
Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-1E
(which utilizes USGS hazard data available in 2008)

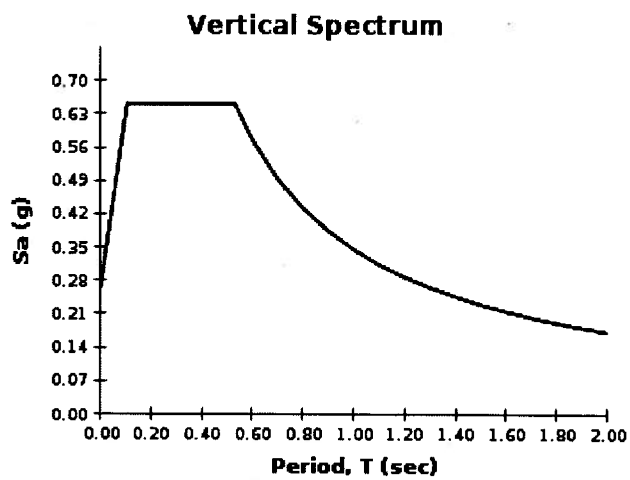
Site Coordinates 37.68545°N, 121.7084°W

Site Soil Classification Site Class D – "Stiff Soil"

USGS–Provided Output

$S_{S,20/50}$	0.841 g	$S_{XS,BSE-1E}$	0.978 g
$S_{1,20/50}$	0.283 g	$S_{X1,BSE-1E}$	0.519 g





Degenkolb

Subject:

Job: B341 INCREMENT II

Job Number: B348910/2.00 Date: 11.8.13

By: AMN Section:

Checked By: Page of

INCREMENT II WEIGHT TAKE OFF

ROOF

TAR & GRAVEL ROOFING (5 PLY)	6 PSF
2" RIGID INSULATION	3 PSF
MTL DECK	3 PSF
M10x9 @ 7'-9" O.C.	1.5 PSF
12WF36 @ 20' O.C.	2 PSF
M/E/P	2 PSF
MISC.	2.5 psf
	20 PSF

$W_{ROOF} = 20\ psf \times 24.33' \times 102'$

$= \underline{\underline{49.6^k}}$

COLUMNS

12WF45 x 19'-5"

$W_{COLS} = 19.42' \times 12 \times \frac{1}{2} \times 45\ plf$

$= \underline{\underline{5.2^k}}$

EXT. CONC. PANELS

6" PANELS = 75 psf HEIGHT = 22'-0"

$W_{PANELS\ LONG.} = 103' \times 22' \times 75\ psf + 9' \times 22' \times 75\ psf$

$= \underline{\underline{185^k}}$

$W_{PANELS\ TRANS} = 2 \times 26.5' \times 22' \times 75\ psf$

$= \underline{\underline{87.5^k}}$

Degenkolb

Subject:

Job: B341 INCREMENT II

Job Number: B31890012.00 Date: 11.8.13

By: AMN Section:

Checked By: Page of

CRANE BM + RAILS

$$14WF48 + \#40 \text{ RAIL} = 48 + 40/3 = 61.3 \text{ plf}$$

↑ POUNDS PER YARD

$$W_{\substack{CRANE\\RAIL}} = (2)(100')(61.3\,plf)$$

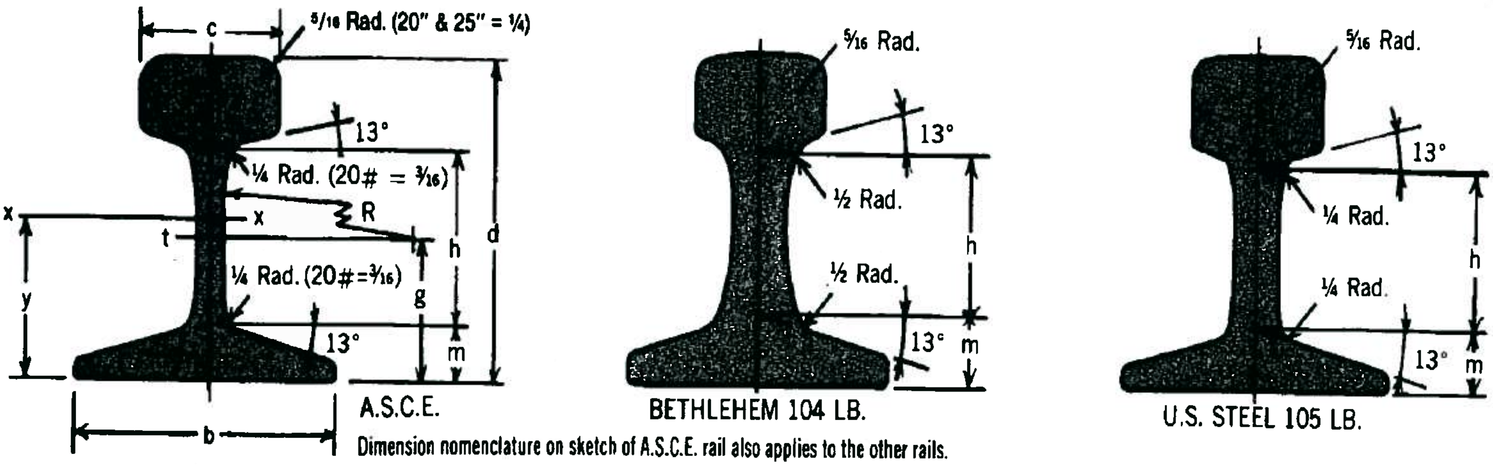
$$= \underline{\underline{12.3^K}}$$

TOTAL WEIGHT

$$W_{TOT} = 49.6^K + 5.2^K + 185^K + 87.5^K + 12.3^K$$

$$= \underline{\underline{386^K}}$$

AMERICAN
CRANE & HOIST



Dimension nomenclature on sketch of A.S.C.E. rail also applies to the other rails.

Type	Desig. (wt. per Yard)	Depth of Section	Base Width	Head Width	Head Radius	Base Thickness	Web Depth	Web Thickness	L Web Radius (Gage)	Web Radius	Area of Section	Base to Neutral Axis	Elastic Properties Axis X-X		
		d	b	c	r	m	h	t	g	R	A	y	I	S-head	S-base
	lb.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.^{2}	in.	in.^{4}	in.^{3}	in.^{3}
A.S.C.E.	20	2 5/8	2 5/8	1 11/32	12	7/16	1 15/32	1/4	1 11/64	12	2.00	1.26	1.93	1.41	1.53
A.S.C.E.	25	2 3/4	2 3/4	1 1/2	12	31/64	1 31/64	19/64	1 15/64	12	2.40	1.33	2.50	1.76	1.88
A.S.C.E.	30	3 1/8	3 1/8	1 11/16	12	1 7/32	1 23/32	21/64	1 25/64	12	3.00	1.52	4.10	2.55	2.69
A.S.C.E.	40	3 1/2	3 1/2	1 7/8	12	5/8	1 55/64	25/64	1 9/16	12	3.94	1.68	6.54	3.59	3.89
A.S.C.E.	60	4 1/4	4 1/4	2 3/8	12	49/64	2 17/64	31/64	1 29/32	12	5.93	2.05	14.60	6.64	7.12
A.S.C.E.	70	4 5/8	4 5/8	2 7/16	12	13/16	2 15/32	33/64	2 3/64	12	6.81	2.22	19.70	8.19	8.87
A.S.C.E.	75	4 13/16	4 13/16	2 15/32	12	27/32	2 35/64	17/32	2 1/8	12	7.33	2.30	22.86	9.10	9.94
A.S.C.E.	80	5	5	2 1/2	12	7/8	2 5/8	35/64	2 3/16	12	7.86	2.38	26.38	10.07	11.08
A.S.C.E.	85	5 3/16	5 3/16	2 9/16	12	57/64	2 3/4	9/16	2 17/64	12	8.33	2.47	30.07	11.08	12.17
A.S.C.E.	90	5 3/8	5 3/8	2 5/8	12	59/64	2 55/64	9/16	2 23/64	12	8.83	2.55	34.39	12.19	13.49
A.S.C.E.	100	5 3/4	5 3/4	2 3/4	12	31/32	3 5/64	9/16	2 1/2	12	9.84	2.73	43.97	14.55	16.11
Crane *1	104	5	5	2 1/2	12	1 1/16	2 7/16	1	2 7/16	3 1/2	10.3	2.21	29.8	10.7	13.5
Crane *2	105	5 3/16	5 3/16	2 9/16	12	1	2 13/32	15/16	2 13/64	12	10.3	2.41	34.4	12.4	14.3
Crane *3	135	5 3/4	5 3/16	3 7/16	14	1 1/16	2 13/16	1 1/4	2 15/32	12	13.3	2.81	50.6	17.2	18.0
Crane *1	171	6	6	4.3	Flat	1 1/4	2 3/4	1 1/4	2 5/8	Vert.	16.8	3.01	73.4	24.5	24.4
Crane *3	175	6	6	4 1/4	18	1 9/64	3 7/64	1 1/2	2 21/32	Vert.	17.1	3.02	70.2	23.5	23.3

*1 Bethlehem *2 U.S. Steel *3 Bethlehem & U.S. Steel.

REFERENCES:
- "Manual of Steel Construction," Eighth (1980) Edition, American Institute of Steel Construction, Inc., New York.
- "Bethlehem Trackwork," Bethlehem Steel Corporation, Bethlehem, Pa., Catalog 2341.
- "C.M.A.A." Specification No. 70, Revised 1975. Crane Manufacturers Association of American, Inc., Pittsburgh, Pa., 1975.

Degenkolb

Subject:		Job Number: B389012.00	Date: 11.13.13
Job: B341 INCREMENT 11		By: AMN	Section:
		Checked By:	Page of

TIER 1 QUICK CHECKS

4.5.3.7 - WALL ANCHORAGE

$T_c = \psi S_{XS} w_p A_p$

$\psi = 1.2$ - LIFE SAFETY

$S_{XS} = 0.98g$

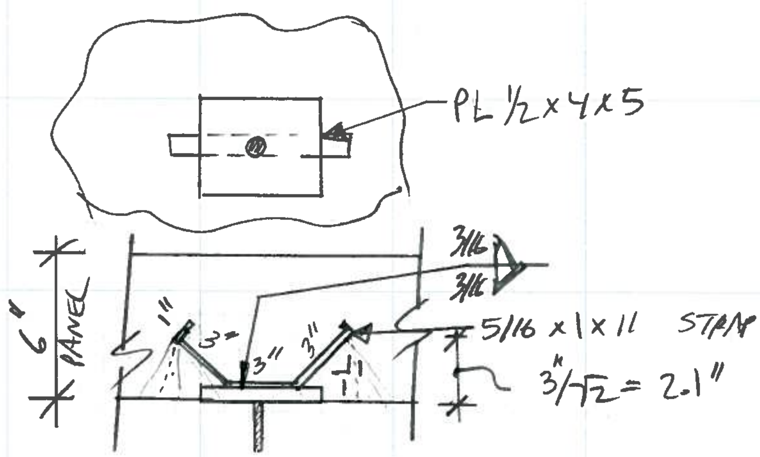
$w_p = 6''/12'' \times 150pcf = 75psf$

$T_c = 1.2 \times 0.98 \times 75psf \times A_p$

$T_c = 88.2 A_p$

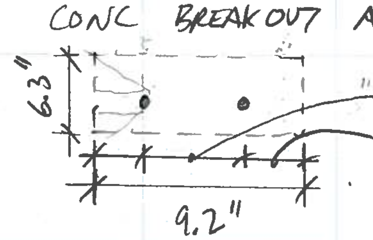
$A_p = \frac{3.58' + 3'}{2} \times \left(\frac{17.33'}{2} + 5'\right) = 45 sf$

$T_c = 88.2 \times 45 = \underline{\underline{4000^{\#}}}$



CALCULATE INSERT CAPACITY PER ACI APPENDIX D
EQUIVALENT TWO ANCHORS ; EMBED = 2.1"

CONC BREAKOUT AREA $= (2)(1.5 \times 2.1'')(3'' + 2 \times 2.1'' \times 2 \times 1'')$
$= 58 in^2$



FROM PROFIS, CAPACITY ($w/ \phi = 1.0$) $= 4136^{\#} > 4000^{\#}$
BUT 0.75 FOR SEISMIC
OK

Degenkoldb

Subject:

Job: B341 INCREMENT 11

Job Number: B3490012.00

Date: 11.13.13

By: AMN

Section:

Checked By:

Page of

CAPACITY OF $\frac{1}{2}''\phi$ THRD STUD

$0.2 \times 36 \times 0.75 = 5.4^k > 4^k$ $\underline{\underline{OK}}$

WELD TO BM – CONT 3/16" FILLET WELD, OK BY INSP.

BENDING OF ANGLE LEG

L4x3x1/4, LLH

$M_u = 4^k \times 1.5'' \times \frac{1}{2} = 3^{k\text{-}in}$

$M_n = 36 \times 4'' \times \frac{0.25^2}{4} \times 1.25 = 2.8^{k\text{-}in} \approx 3^{k\text{-}in}$ $\underline{\underline{OK}}$

$\underline{\underline{\text{PANEL CONNECTION OK}}}$

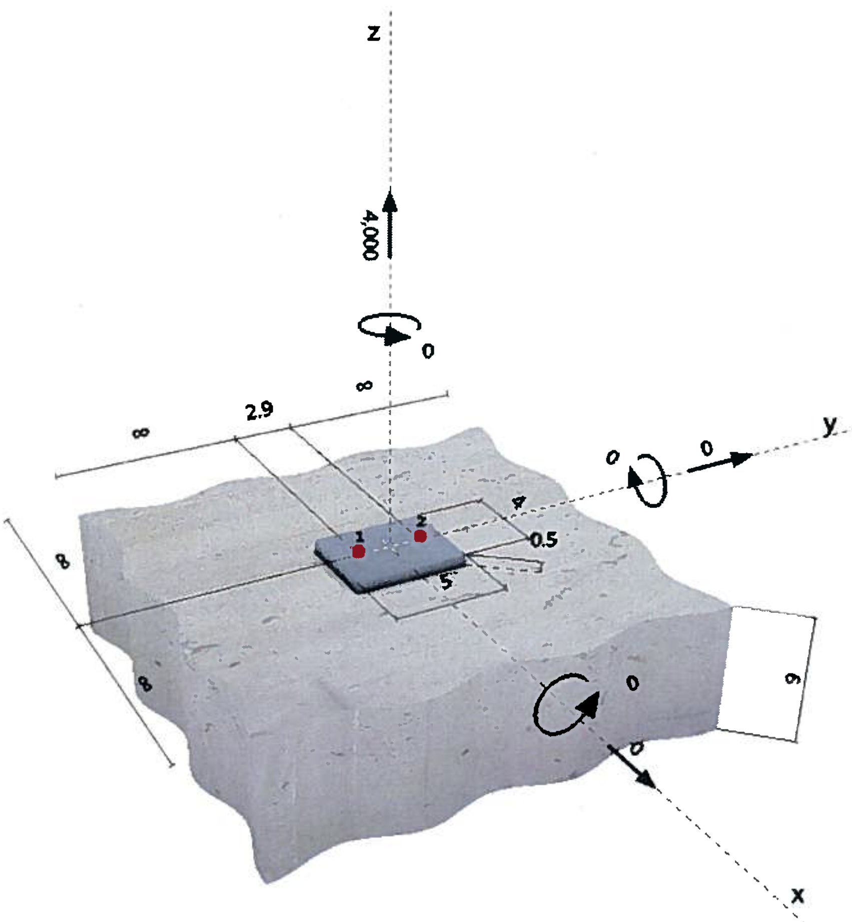
Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	11/13/2013
E-Mail:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ - CS 1/2 (2)**
Effective embedment depth:	h_{ef} = 2.000 in., h_{nom} = 2.375 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-1917
Issued I Valid:	5/1/2013 | 5/1/2015
Proof:	design method ACI 318 / AC193
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 4.000 in. x 5.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	uncracked concrete, 3000, f_c' = 3000 psi; h = 6.000 in.
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



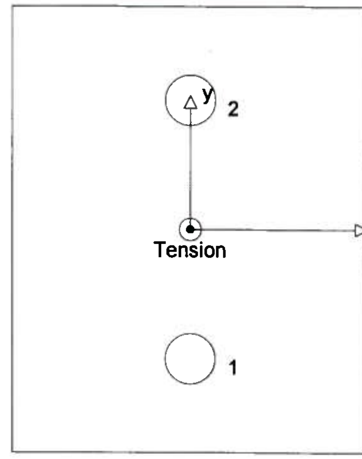
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2000	0	0	0
2	2000	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 4000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2000	8029	25	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	4000	3585	112	not recommended

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-1917
$\phi\, N_{steel} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

n	$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1	0.10	106000

Calculations

N_{sa} [lb]
10705

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
10705	0.750	8029	2000

3.2 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-08 Eq. (D-5)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-08 Eq. (D-1)
A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-08 Eq. (D-6)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-08 Eq. (D-9)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-08 Eq. (D-11)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-08 Eq. (D-13)
$N_b = k_c\ \lambda\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
4.500	24	1	3000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
53.40	36.00	1.000	1.000	1.000	1.000	3718

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
$\boxed{5515}$	0.650	3585	4000

CONC. CAPACITY $\phi = 1.0$

MULTIPLY BY 0.75 FOR SEISMIC

$V_c = 4136^{\#}$

Degenkolb

Subject:		Job Number: B3199 02.00	Date: 11-13-13
Job:	B341 INCREMENT II	By: AMN	Section:
		Checked By:	Page of

4.5.3.3 SHEAR STRESS CHECK

$V = C S_a W \qquad C = 1.4 \qquad S_a = S_{XS} = 0.98$

$V = (1.4)(0.98)W = \underline{\underline{1.37W}}$

LONG DIR

$V = \left[\left(\frac{1}{2}\right)(50^k + 5.2^k + 88^k + 12.3^k) + 185^k\right] 1.37$

$V = 360^k$

$v_{avg} = \frac{1}{M_s} \frac{V}{A_w} = \frac{1}{4} \frac{360^k}{(6'' \times 94' \times 12''/')} = 0.0133 \text{ ksi}$

$= 13.3 \text{ psi} < 2\sqrt{f'_c}$

OK

TRANS. DIR

MOMENT FRAMES @ 20' O.C.

ASSUME END WALLS ONLY RESIST 10' TRIB WIDTH

$V = \left[\frac{1}{2}(88^k) + \frac{10'}{100'} \times (50^k + 5^k + 185^k + 12^k)\right] 1.37$

$V = 95^k$

$v_{avg} = \frac{95^k}{4} \times \frac{1}{(2 \times 6.75' \times 12''/' \times 6'')} = 0.024^{ksi}$

$= 24 \text{ psi} < 2\sqrt{f'_c}$

OK

![Degenkolb]

Subject:		Job Number: B3189012.00	Date: 11.13.13
Job: B341 INCREMENT II		By: AMN	Section:
		Checked By:	Page of

REINFORCING STEEL RATIO

6" PANEL w/ #4 @ 12" O.C., E.F.

$\rho_H = \rho_v = 0.2/6\times12 = 0.0028 > 0.0012$ OK

> 0.002 OK

MOMENT FRAME DRIFT CHECK

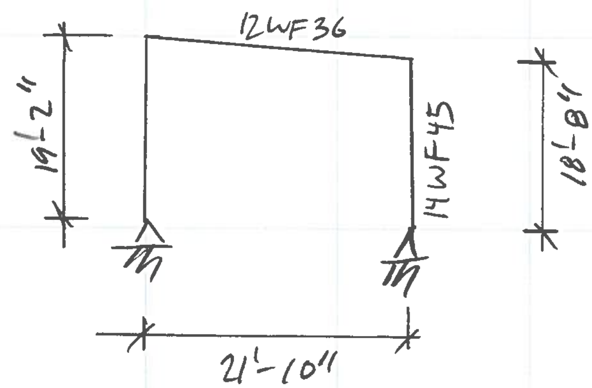
$V = C\,S_a\,W$ $\qquad C = 1.3$

$\underline{\underline{V = 1.27\,W}}$ $\qquad S_a = S_{XS} = 0.98$

TYP. MF @ 20' O.C.

$$V = \frac{20'}{100'} \times \left(50^k + 5^k + \tfrac{1}{2}(185^k) + 12^k\right)(1.27)$$

$$= 40.5^k$$



FROM ETABS MODEL, DRIFT = 12.2"

DRIFT RATIO $= 12.2''/18.92' \times 12''/' = 0.054 > 0.025$

NO

ACTUAL PERIOD $= 2\pi\sqrt{M/K}$ $\qquad M = 40.5^k/1.27/386.4 = 8.25\times10^{-2}\ \text{k-s}^2/\text{in}$

$\qquad K = 40.5^k/12.2'' = 3.3^k/\text{in}$

$$= 2\pi\sqrt{\frac{8.25\times10^{-2}}{3.3}}$$

$= 1$ SEC $\qquad \therefore\ S_A = 0.52g\ \ S_{X1}$

$V = 40^k \times 0.52g/0.98g = 21^k$

$DR = \frac{21^k}{40^k} \times 0.054 = 0.028 \approx 0.025$ OK

Degenkolb

Subject:	Job Number: B3189012.00	Date: 11.13.13
Job: B341 INCREMENT 11	By: AMN	Section:
	Checked By:	Page of

MOMENT FRAME AXIAL STRESS CHECK

$$P_{OT} = \frac{1}{M_S} \times V \times \frac{h}{L} \times \frac{1}{A_{COL}} \qquad M_S = 2.0$$

$$= \frac{1}{2} \times 40.5^k \times \frac{18.92'}{21.83'} \times \frac{1}{13.2\,in^2} = 1.3^{ksi} < 0.3 \times 36^{ksi} = 10.8^{ksi}$$

$\underline{\underline{OK}}$

MOMENT FRAME FLEXURAL STRESS CHECK

FROM ETABS MODEL

$$M_{u_{COL}} = 365^{k\text{-}ft}$$

$$M_{u_{BM}} = 367^{k\text{-}ft}$$

$$\sigma = \frac{M_u}{Z M_S} \qquad M_S = 8.0$$

$$\sigma_{COL} = \frac{365^{k\text{-}ft} \times 12''/'}{(8)(69.6\,in^3)} = 7.9^{ksi} < 36^{ksi} \quad \underline{\underline{OK}}$$

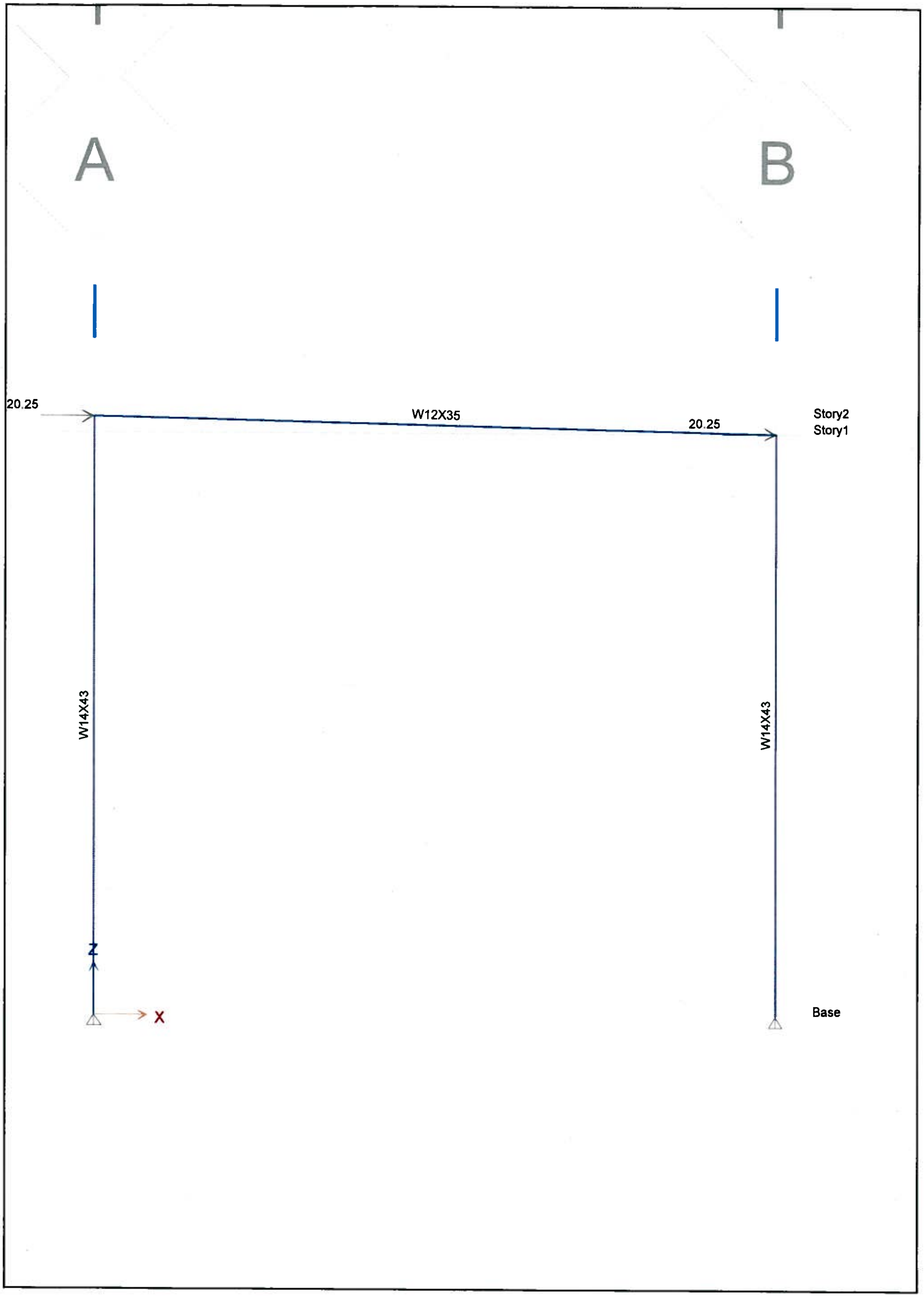
$$\sigma_{BM} = \frac{367^{k\text{-}ft} \times 12''/'}{(8)(51.2\,in^3)} = 10.8^{ksi} < 36^{ksi} \quad \underline{\underline{OK}}$$

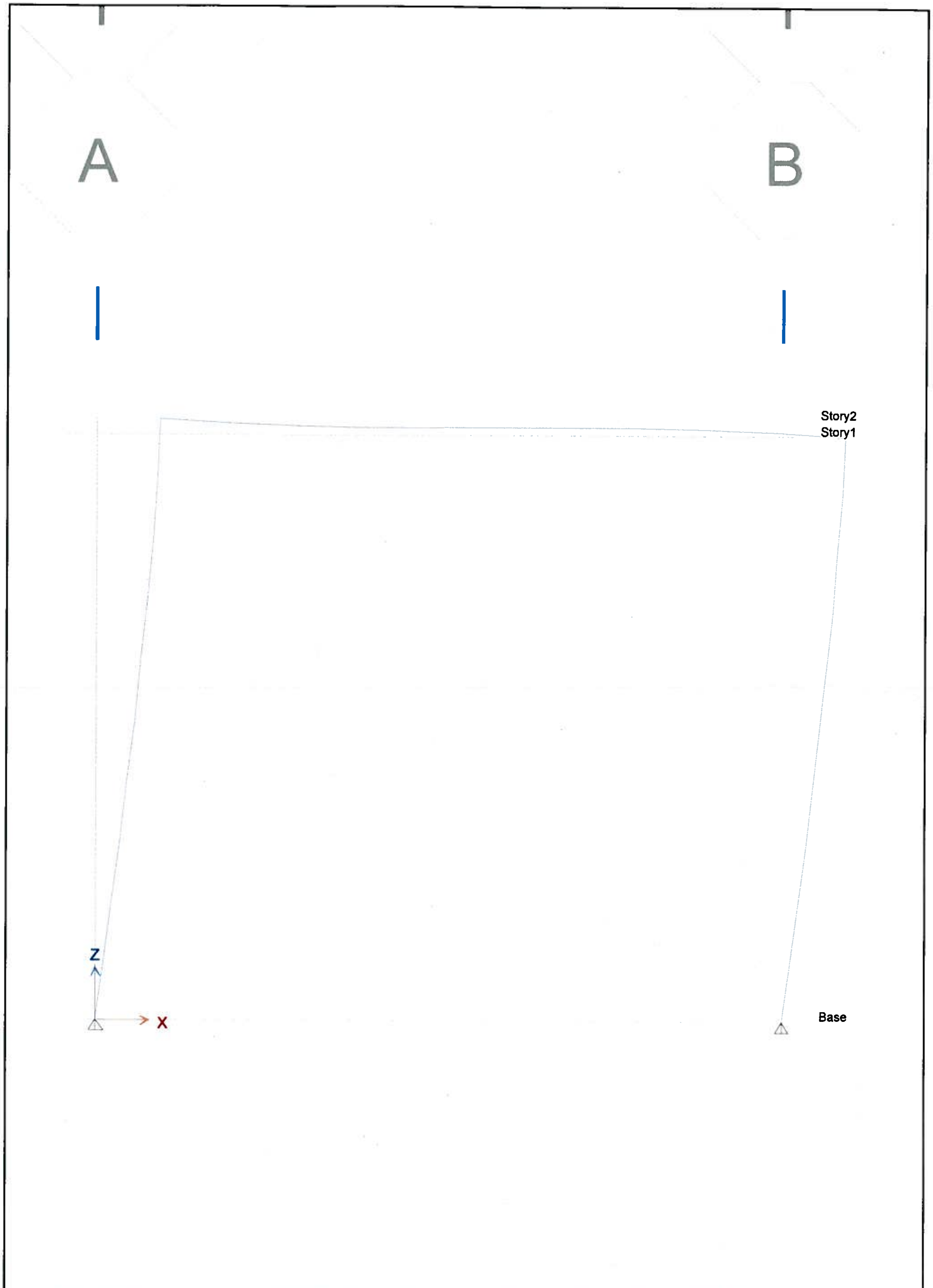
USING REVISED PERIOD OF 1 SEC

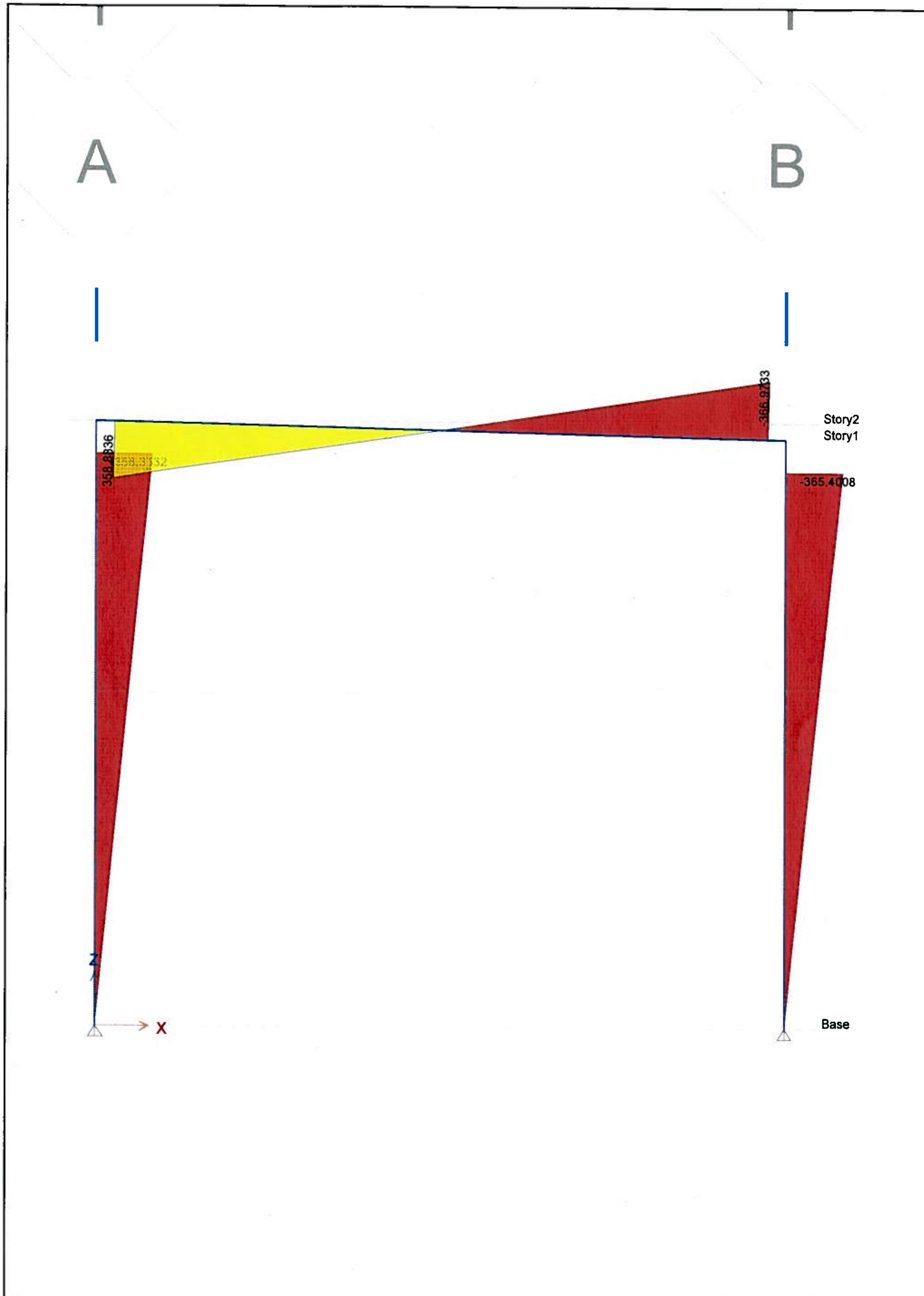
$$M_{u_{BM}} = \frac{0.52g}{0.98g} \times 367^{k\text{-}ft} = 195^{k\text{-}ft}$$

$$\sigma_{bm} = 195/51.2 \times 12 = 46^{ksi} \approx \text{ELASTIC}$$

$$\sigma_{COL} = 365^{k\text{-}ft} \times 0.52/0.98 / 69.6 \times 12 = 33^{ksi} = \text{ELASTIC}$$


A
B
20.25
W12X35
20.25
Story2
Story1
W14X43
W14X43
Z
X
Base


A
B
Story2
Story1
Z
X
Base


A
B
358.8836
-366.9733
-365.4008
Story2
Story1
Base
Z
X

Degenkolb

Subject:
Job: B341 INCREMENT II
Job Number: B3189012.00
Date: 11.13.13
By: AMN
Section:
Checked By:
Page of

MOMENT FRAME PANEL ZONES

$M_{p_{BM}} = 1.25 \times 36 \times 51.2\,in^3 = 2304^{k\text{-}in}$

$V_{\substack{PANEL \\ ZONE}} = \dfrac{2304^{k\text{-}in}}{(12.5-0.5)} = 192^k$

$V_{n_{\substack{PANEL \\ ZONE}}} = (0.6)(1.25\times36)(0.31''\times13.66'')$ $\quad t_{WEB} = 0.31''$

$= 114^k < 0.8\times192^k = 154^k$ $\quad d_{COL} = 13.66''$

NO → TIER 3 ↗

ACTUAL $M_{u_{BM}} = 195^{k\text{-}ft} = 2340^{k\text{-}in}$

$m_{\substack{PANEL \\ ZONE}} = 8$, TABLE 9-5

$V_u = \dfrac{2340^{k\text{-}in}}{8(12'')} = 24^k$

$V_n = 114^k > 24^k$

OK

STRONG COL. WEAK BM

$Z_{BM} = 51\,in^3 < Z_{COL} = 69\,in^3$ OK

COMPACT MEMBER CHECK

MODERATELY DUCTILE MEMBERS

BM/COL FLANGE $\quad b/t \leq 0.38\sqrt{E/F_y} = 0.38\sqrt{\dfrac{29000}{45}}$

≤ 9.6

12WF36 $\quad b/t = \dfrac{6.56''}{(2)(0.52'')} = 6.3$ OK

14WF45 $\quad b/t = \dfrac{8''}{(2)(0.53'')} = 7.5$ OK

BM/COL WEBS $\quad h/t_w \leq 1.49\sqrt{E/F_y} = 37.8$

12WF36 $\quad h/t_w = \dfrac{10.125''}{0.3} = 33.8$ OK

14WF45 $\quad h/t_w = \dfrac{10.875''}{0.31''} = 35.1$ OK

Degenkolb

Subject:
Job:
Job Number: Date:
By: Section:
Checked By: Page of

CHECK BM FLEXURAL DEMAND @ PANEL ZONE CAPACITY?

BM-COL CONN IS COVER PLATE CONN.

$V_{PANEL} = 114^{K}$

$M_{u_{BM}} = 12'' \times 114^{K} = 1368^{k-in} / 12''/' = 114^{k-ft}$

$\sigma_{BM \text{ @PANEL CAP}} = \dfrac{1368^{k-in}}{51.2 \text{ in}^3} = 27^{ksi}$

$\therefore \; 27^{ksi}/44^{ksi} = 0.6 F_{ye}$

$F_{FLNG} = 27^{ksi} \times 6.56'' \times 0.52'' = 92^{K}$

$A_{REQD} = 92^{k}/(1.25 \times 36) = 2 \text{ in}^2$

$\text{WELD @ } 5/16'' \text{ FILLET} = 92^{k} / \left(\frac{1.39}{0.75} \times 5\right)(2) = 5''$

EA SIDE

BASED ON VISUAL OBSERVATION CONN. APPEARS TO HAVE ADEQUATE CAP.

Degenkolb

Subject:		Job Number: B3189012.00	Date: 11.14.13
Job: B341 INCREMENT 11		By: AMN	Section:
		Checked By:	Page of

BRACED FRAME AXIAL STRESS CHECK - COLS.

$V = C S_a W$

$S_a = S_{xs} = 0.98$

$C = 1.4$

$\underline{V = 1.37W}$

$V = \frac{1}{2}(50^k + 5.2^k + \frac{1}{2}88^k + 12^k)1.37$

ASSUME CONC PANELS CARRY OWN LOAD IN LONG DIR.

$V = 76^k$ PER LINE

2 BAYS PER LINE

$\sigma_{COL} = \frac{76^k}{2} \times \frac{19.6'}{20'} \times \frac{1}{13.2 in^2} \times \frac{1}{2}$

$m_s = 2$

$= 1.4^{ksi} < 0.3F_y$ OK

BRACED FRAME AXIAL STRESS CHECK - BRACES

DOUBLE ANGLE 2- L2x2x1/4

$L_{BRACE} = \sqrt{20^2 + 18.75^2}' = 27.4'$

$A_{BRACE} = 1.89 in^2$ $r = 0.842''$ $\frac{Kl}{r} = \frac{27.4' \times 12 \times (1)}{0.842} = 390$

→ TENSION ONLY

$m_s = 3.0$

$\sigma_{BRACE} = \frac{76^k \times 27.4'}{3 \times 20' \times 2 \times 1.89 in^2}$

$\sigma_{BRACE} = 9^{ksi} < 0.5F_y = 18^{ksi}$

OK

Degenkolb

Subject:
Job: B341 INCREMENT 11
Job Number: B31890/2.00
Date: 11.14.13
By: AMN
Section:
Checked By:
Page of

BRACE FRAME CONNECTION STRENGTH

$\frac{kl}{r} = 390 \longrightarrow F_e \approx 1.7\text{ ksi}$

$A_{BRACE} = 1.89\text{in}^2$

$P_{cr} = 1.7^{ksi} \times 1.89\text{in}^2 = 3.2^k$ $\quad P_{ye} = 1.25 \times 36 \times 1.89 = 85^k$

CONN CAPACITY $\approx 2 \times \frac{1.39}{0.75}^{k/in/16TH} \times 3 \times 2'' = 22^k > 3.2^k$

$< 85^k$

3/16" FILET WELD EA SIDE 2" LONG

NO

BRACE COMPACT MEMBERS

$L2 \times 2 \times 1/4$ $\quad B/t = 2/0.25 = 8 < 0.38\sqrt{\frac{E}{F_y}} = 9.6$

OK

Appendix B: Increment 3 Tier 1 Check Lists and Calculations

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment III | Date: November 14, 2013
Building Address: Lawrence Livermore National Laboratory | Page: 1 of 2
Job Number: B3189012.00 | Job Name: LLNL B341 Seismic Evaluation | By: AMN | Checked: ____

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments

LOW SEISMICITY

BUILDING SYSTEM

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Increment III is attached to Increment I, however, given that the buildings are relatively stiff, we do not believe there will be significant relative movement at this location, therefore we do not believe this is a life-safety concern.**
☐	☐	☒	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

BUILDING CONFIGURATION

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☐	☐	☒	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)

GEOLOGIC SITE HAZARDS

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment III | Date: November 14, 2013

Building Address: Lawrence Livermore National Laboratory | Page: 2 of 2

Job Number: B3189012.00 | Job Name: LLNL B341 Seismic Evaluation | By: AMN | Checked: ______

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	
☐	☐	☒	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	**30'/12.5' = 2.5 > 0.6*0.98 = 0.59**
☐	☐	☒	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	**Foundation tied by slab-on-grade.**

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment III Date: November 14, 2013

Building Address: Lawrence Livermore National Laboratory Page: 1 of 2

Job Number: B3189012.00 Job Name: LLNL B341 Seismic Evaluation By: AMN Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C NC N/A U Comments

LOW AND MODERATE SEISMICITY

Seismic-Force-Resisting System

☐ ☐ ☒ ☐ COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)

☒ ☐ ☐ ☐ REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)

☒ ☐ ☐ ☐ SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)

☒ ☐ ☐ ☐ REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)

Connections

☐ ☐ ☒ ☐ WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

☒ ☐ ☐ ☐ TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)

☒ ☐ ☐ ☐ FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

☐ ☐ ☒ ☐ DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

☐ ☐ ☒ ☐ FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)

☐ ☐ ☒ ☐ COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)

Degenkolb
DEGENKOLB ENGINEERS

Building Name: 341, Increment III | Date: November 14, 2013
Building Address: Lawrence Livermore National Laboratory | Page: 2 of 2
Job Number: B3189012.00 | Job Name: LLNL B341 Seismic Evaluation | By: AMN | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Degenkolb

Subject:
Job: B341 INCREMENT III
Job Number: B3189012.00
Date: 11.14.13
By: AMW
Section:
Checked By:
Page of

INCREMENT III WEIGHT TAKEOFF

TAR & GRAVEL ROOFING (5 PLY)	6 PSF
1 5/8" RIGID INSULATION	2 psf
	8 psf

WOOD ROOF

2x12 @ 16" O.C.	3.5 psf
1/2" PLYWD	1.7 psf
3 1/2" BATT INSUL	0.5 psf
5/8" GYP BOARD	2.8 psf
MEP	3 psf
MISC	2 psf
	14 psf

CONC. ROOF

8" CONC. SLAB	100 psf
MEP	3 psf
MISC.	2 psf
	105 psf

TOTALS:

CONC ROOF	113 psf
WOOD ROOF	22 psf

EXTERIOR STUD WALLS

2x4 @ 16" O.C.	1.1 psf
5/8" GYP BOARD	2.8 psf
BATT INSULATION	0.5 psf
STUCCO	10 psf
MEP + MISC.	1
	15 psf

Degenkolb

Subject:		Job Number: B3189012.00	Date: 11.14.13
Job: B341 INCREMENT III		By: AMN	Section:
		Checked By:	Page of

CONC. WALLS

8" CONC.	100 psf
PLASTER SKIM COAT	2 psf
MEP + MISC.	1 psf
	103 psf

BUILDING WEIGHT

WOOD ROOF $= 22\,psf \times [(2)(18.66')(39') + (38.66' \times 12')]$

$= \underline{\underline{42^K}}$

CONC. ROOF $= 105\,psf \times 38.66' \times 27' = \underline{\underline{110^K}}$

STUD WALLS $= 15\,psf \times [2 \times (39' + 18.66') \times 13.62' + 39' \times 12.5']$

$= \underline{\underline{31^K}}$

CONC. WALLS $= 103\,psf \times [76' + 54'] \times 11.83' = \underline{\underline{158^K}}$

TOTAL $= 42^K + 110^K + 31^K + 158^K$

$= \underline{\underline{341^K}}$

Degenkolb

Subject:		Job Number: B3189012.00	Date: 11.14.13
Job: B341 INCREMENT III		By: AMN	Section:
		Checked By:	Page of

PSEUDO LATERAL LOAD

$V = C S_a W$ $\qquad S_a = S_{xs} = 0.98g$

$V = 1.37W$ $\qquad C = 1.4$

$= 1.37 \times 341^k$

$= 467^k$

TIER 1 QUICK CHECKS

CONC. WALL SHEAR STRESS CHECK

EAST-WEST DIR CONTROLS

$$\sigma = \frac{467^k}{4 \times 8'' \times (2)(38' - 1.5' - 6' - 3')(12''/')} \qquad M_s = 4.0$$

$= 0.022^{ksi} = 22\,psi < 2\sqrt{3000} = 110\,psi$ OK

REINFORCING RATIO

VERT: #6 @ 8" $\rho_V = 0.44\,in^2/(8)(12) = 0.0046 > 0.0012$

HOR: #5 @ 8" $\rho_H = 0.31\,in^2/(8)(12) = 0.0032 > 0.002$

OK

Appendix C: Increment 1 Tier 3 Calculations

Lawrence Livermore National Laboratory
Building 341 Detailed Seismic Evaluation
Structural Calculations

Seismic Weight Takeoff and Lateral Loading

December 2013

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject:	Seismic Weight Take-off	**Job Number:** B3189012.	**Date:** 11.14.13
Job:	LLNL B341 Increment I	**By:** AMN	**Section:**
		Checked By:	

Building Weight

High Roof

Component	Deck	Beam	Girders	Columns	Lateral
5 ply Tar and Gravel Roofing	6	6	6	6	6
Metal Deck	3	3	3	3	3
10B11.5 @ 7'-2" o.c.	0	1.6	1.6	1.6	1.6
21WF68 @ 20' o.c.	0	0	3.4	3.4	3.4
Insulation	3	3	3	3	3
MEP		5	5	5	5
Misc.		2	2	2	2
Total	**12**	**21**	**24**	**24**	**24**

Low Roof

Component	Deck	Beam	Girders	Columns	Lateral
5 ply Tar and Gravel Roofing	6	6	6	6	6
Metal Deck	3	3	3	3	3
10B15 @ 7'-6" o.c.	0	2.0	2.0	2.0	2.0
21WF55 & 33WF152 @ 20' o.c.	0	0	5.2	5.2	5.2
Insulation	3	3	3	3	3
MEP		5	5	5	5
Misc.		2	2	2	2
Total	**12**	**21**	**26**	**26**	**26**

Equipment Loft

Component	Deck	Beam	Girders	Columns	Lateral
6.5" concrete slab	81	81	81	81	81
12WF40 @ 7'-6"	0	7.2	7.2	7.2	7.2
Conc bm @ line 2a	0	9	9	9	9
33WF152 @ 20' o.c.	0	0	8	5	5
MEP	21	21	21	21	21
Misc		5	5	5	5
Total	**103**	**124**	**131**	**129**	**129**

Mezzanine with conc beams

Component	Deck	Beam	Girders	Columns	Lateral
6.5" concrete slab	81	81	81	81	81
2'x1'-3" bm @ 10'-0" +/- o.c.	0	38	38	38	38
16WF45	0	2	2	2	2
MEP		10	10	10	10
Misc		5	5	5	5
Total	**81**	**135**	**135**	**135**	**135**

Mezzanine with steel beams

Component	Deck	Beam	Girders	Columns	Lateral
6.5" concrete slab	81	81	81	81	81

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject:	Seismic Weight Take-off	**Job Number:**	B3189012.	**Date:**	11.14.13
Job:	LLNL B341 Increment I	**By:**	AMN	**Section:**	
		Checked By:			

16WF35 @ 7'-4" +/-	0	8	8	8	8
24WF76 @ 20' o.c.	0	0	4	4	4
MEP		10	10	10	10
Misc		5	5	5	5
Total	**81**	**104**	**108**	**108**	**108**

Concrete Wall Weights

	Weight (psf)	Length (ft)	Height (ft)	Area (sf)	Total Weight (k)
Precast Concrete Panels: High Roof (6")	75	280	32.5	9100	683
Precast Concrete Panels: Low Roof (6")	75	360	22	6732	505
Precast Concrete Panels: Bet. Low and High R	75	40	12	460	35
Cast-in-place conc. Walls: High Roof (8")	100	180	31	5580	558
Cast-in-place conc. Walls: Mezzanine (8")	100	280	13	3640	364
		1140			
				Total =	**2144**

Total Building Weight

Floor	Floor Area (sf)	Floor Weight (k)	Wall Weight (k)	Total Weight (k)	Total Weight (psf)	Total Mass (lbs-s2/ft3)
High Roof	11345	272	638	910	80	2.491
Low Roof	13855	363	270	632	46	1.417
Equipment Loft	2345	302	0	302	129	3.997
Mezzanine with conc. bms	2733	370	182	552	202	6.274
Mezzanine with steel bms	2266	245	0	245	108	3.354

Total Floor Weight = **1552** **kips**
Total Wall Weight = **2144** **kips**

Total Building Weight = **3696** **kips**

SAP Assembled Joint Mass = **3662** **kips** 99%

Degenkolb

Subject:		Job Number: B3189012.00	Date: 11.19.13
Job: B341 INCREMENT I		By: AMN	Section:
		Checked By:	Page of

ASCE 41-13 LINEAR DYNAMIC PROCEDURE

- SCALE BSE-1E RESPONSE SPECTRUM BY $C_1 C_2$
- PER TABLE 7-3 FOR m BETWEEN 2 & 6

$C_1 C_2 = 1.4$

USGS Design Maps Summary Report

User–Specified Input

Report Title LLNL - Building 341
Wed November 13, 2013 18:35:49 UTC

Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-1E
(which utilizes USGS hazard data available in 2008)

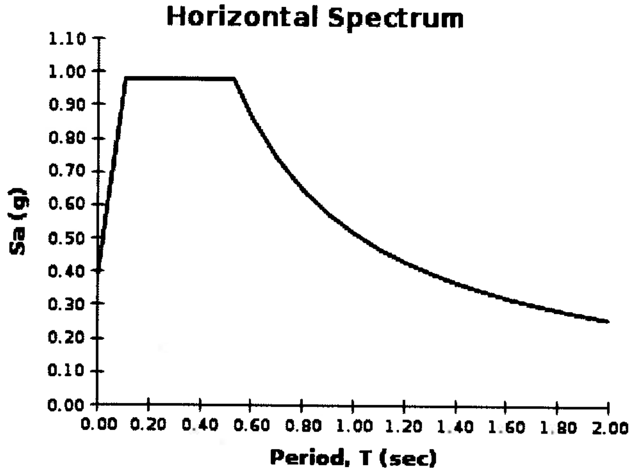
Site Coordinates 37.68545°N, 121.7084°W

Site Soil Classification Site Class D – "Stiff Soil"

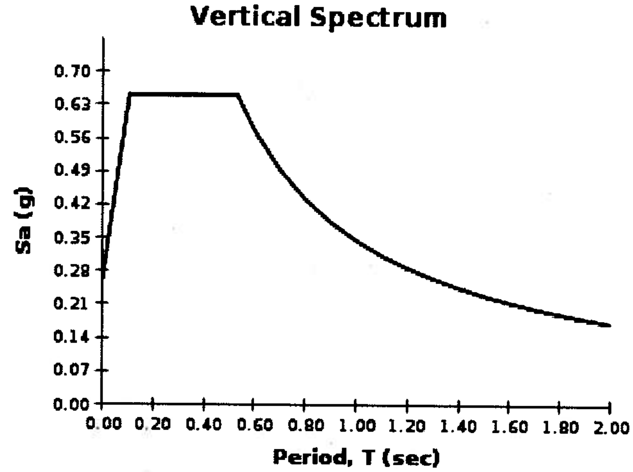
USGS–Provided Output

$S_{S,20/50}$	0.841 g	$S_{XS,BSE-1E}$	0.978 g
$S_{1,20/50}$	0.283 g	$S_{X1,BSE-1E}$	0.519 g

Horizontal Spectrum



Vertical Spectrum



Degenkolb

Degenkolb Engineers
1300 Clay Street
Oakland, CA 94612
Phone: 510.272.9040
Fax: 510.272.9526

Subject: ASCE 41-13 Lateral Load	**Job Number:** B3189012.00	**Date:** 10.31.2013	
Job: LLNL B341 Increment 1	**By:** AMN	**Section:**	
	Checked By:	**Page**	**of**

ASCE 41-13 Pseudo Lateral Load

INPUT DATA

S_s:	Short Period Response Acceleration =	1.47	x g
S_1:	Spectral Response Acceleration @ 1 sec. =	0.52	x g
SC:	Soil Class =	D	
C1*C2:	C1*C2	1.4	
W:	Total Building Weight =	3696	kips
h:	Total Building Height =	31	feet
A_b:	Base Area of Building =	2733	ft^2
C_t:	Building System Coefficient =	0.02	
x:	Period Determination Exponent =	0.75	
n:	Number of Stories =	2	
T_L:	Long-Period Transition Period =	12	sec.

CALCULATE BASE SHEAR

	Controlling Sesimic Design Category =	D	
T_a:	Approximate Fundamental Period of Vibration = $C_t * h_n^x$ =	0.26	sec.
T:	Calculated Structural Period =	0.26	sec.
T:	Limiting Structural Period =	0.37	sec.
T:	Design Structural Period =	0.26	sec.
k:	Vertical Force Distribution Exponent =	1.00	

Table 1615.1.2(2):	S_1 <= 0.1	S_1 = 0.2	S_1 = 0.3	S_1 = 0.4	S_1 >= 0.5
Soil Class D	2.4	2.0	1.8	1.6	1.5
F_v	-	-	-	-	1.50

F_v:	Site Coefficient for S_1 =	1.50	

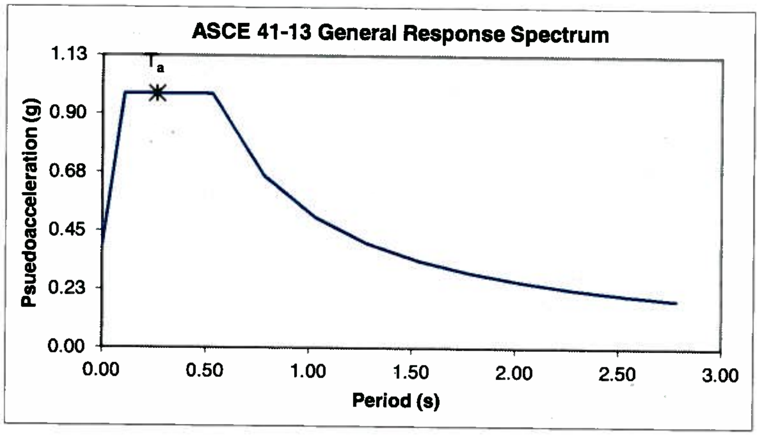
Table 1615.1.2(1):	S_s <= 0.25	S_s = 0.50	S_s = 0.75	S_s = 1.00	S_s >= 1.25
Soil Class D	1.6	1.4	1.2	1.1	1.0
F_a	-	-	-	-	1.00

F_a:	Site Coefficient for S_s =	1.00	
S_{MS}:	Modified Short Period Acceleration = F_a*S_s =	1.47	
S_{M1}:	Modified 1 sec. Period Acceleration = F_v*S_1 =	0.78	
S_{DS}:	Design Short Period Acceleration = 2/3*S_{MS} =	0.98	
S_{D1}:	Design 1 sec. Period Acceleration = 2/3*S_{M1} =	0.52	
T_s:	$T_S = S_{D1}/S_{DS}$ =	0.53	sec.
T_0:	$T_0 = 0.2T_S$ =	0.11	sec.
S_a:	Design Spectral Acceleration at Building Period =	0.98	
C1C2Sa	C1*C2*Sa =	1.372	**<= Controls**
V:	Pseudo Lateral Load = C1*C2*Sa*W =	$\boxed{5070}$	**kips**

Degenkolb

Degenkolb Engineers
1300 Clay Street
Oakland, CA 94612
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	ASCE 41-13 Lateral Load	**Job Number:**	B3189012.00	**Date:**	10.31.2013
Job:	LLNL B341 Increment 1	**By:**	AMN	**Section:**	
		Checked By:		**Page**	**of**



CALCULATE VERTICAL FORCE DISTRIBUTION

Story	w_x (kips)	h_x (feet)	$w_x{}^*h_x{}^k$	C_{vx}	F_x (kips)	V_x (kips)
High Roof	910	31.25	28432	0.48	2449	2449
Low Roof	934	21.5	20084	0.34	1730	4178
Mezz	797	13	10359	0.18	892	5070
			58875	1.00		5070

CALCULATE DIAPHRAGM FORCE DISTRIBUTION

Story	w_x (kips)	F_x (kips)	Fpx (kips)	Fpx min (kips)	Fpx max (kips)	Diaphragm Design Force (kips)	Fpx/Fx
High Roof	910	2449	2449	178	357	2449	1.00
Low Roof	934	1730	2117	183	366	2117	1.22
Mezz	797	892	1530	156	312	1530	1.71

Degenkolb

Subject: FLOOR/SLAB ELEVS
Job: LLNL - B341

Job Number:
By:
Checked By:
Date:
Section:
Page of

ELEVATIONS - INCREMENT 1

TYP. BOT. OF FTG.	609'-6" TO	611'-0"
T.OP OF S.O.G.	615'-0"	0'-0"
MEZZ. FLR SLAB (E-K/1-2)	628'-0"	13'-0"
EQUIPMENT LOFT (B-J/2-2a)	636'-6"	21'-6"
LOW ROOF (A-K/2-4)	635'-0"	20'-0"
HIGH ROOF (A-K/1-2)	646'-0"	31'-0"

ELEVATIONS - INCREMENT 2 (SEPARATE STRUCT)

T.O. S.O.G.	615'-0"	0'-0"
T.O. STL ROOF		19'-2"

ELEVATIONS - INCREMENT 3 (TIED INTO INCR. 1)

T.O. S.O.G.	615'-0"	0'-0"
T.O. CONC. ROOF SLAB		12'-2"

SAP 2000 3D Model Development

December 2013

REVISION 1, 12-4-84

PRELIMINARY SEISMIC EVALUATON FOR B-341

A. Description of the building structures:

The main building is a 140' x 180' rectangular shaped one story building with a 50' x 100' mezzanine floor at the south-east corner. It is constructed of a steel frame with precast concrete panels attached to the perimeter of the steel frame, and with steel decks on the top of the frame as the roof.

It, basically, is a "Box System" structure. The lateral loads i.e., wind or seismic loads are laterally supported by the roof diaphragms, then carried into shear walls, the precast concrete wall panels and a few internal cast-in-place concrete walls. The steel frame is designed for vertical loads only, therefore, there are no vertical bracing systems, and some horizontal bracing systems under the roof are designed for construction purpose only.

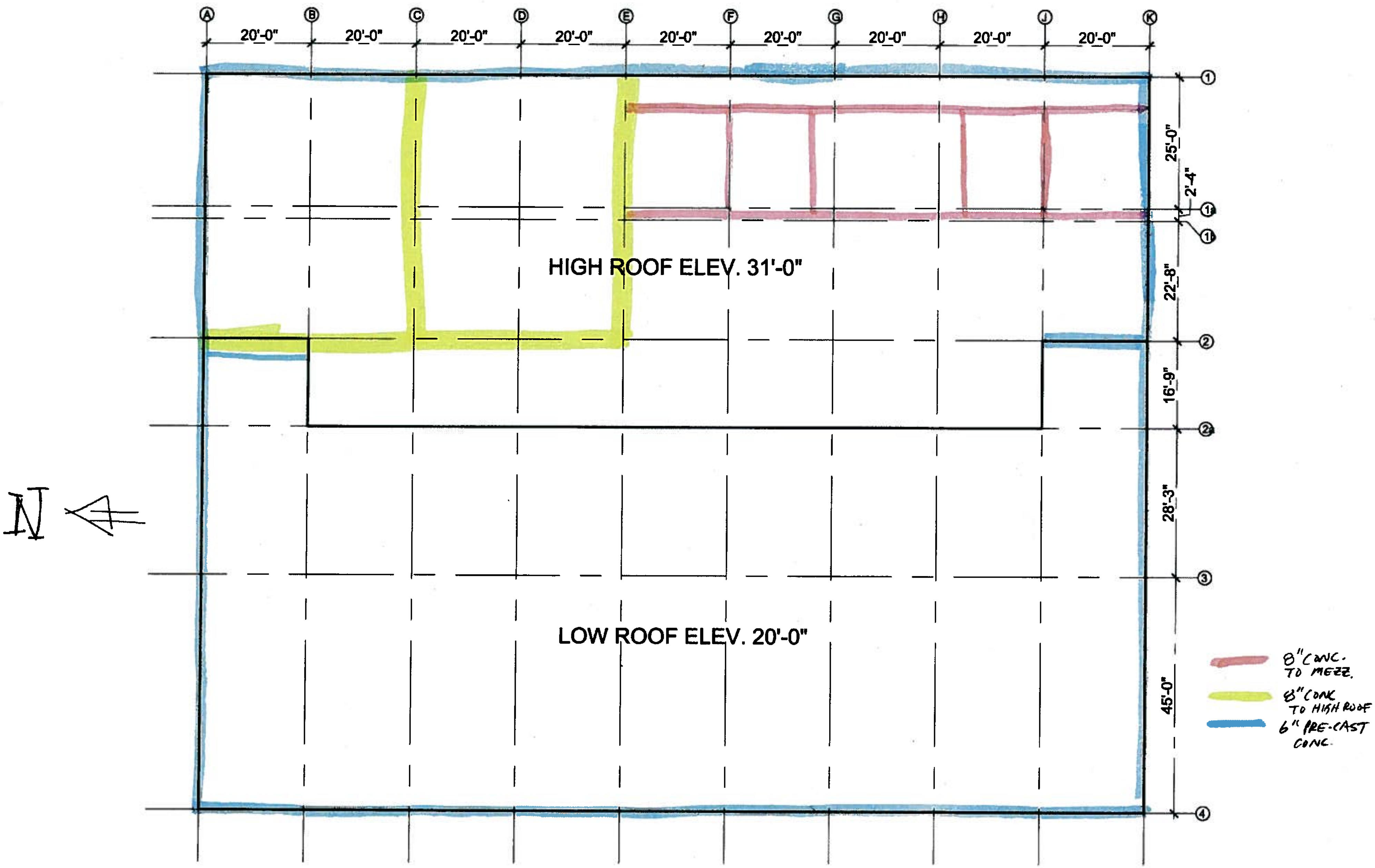
The roof system is split into two levels stepping down from east to west, (see sketch 1). The high bay is 50' wide, and 31' above the grade slab. The low roof, is 90' wide and 20' above the slab. A 17' wide strip of the low roof, adjacent to the high bay, is utilized as a mechanical loft covered with light gauge steel panels.

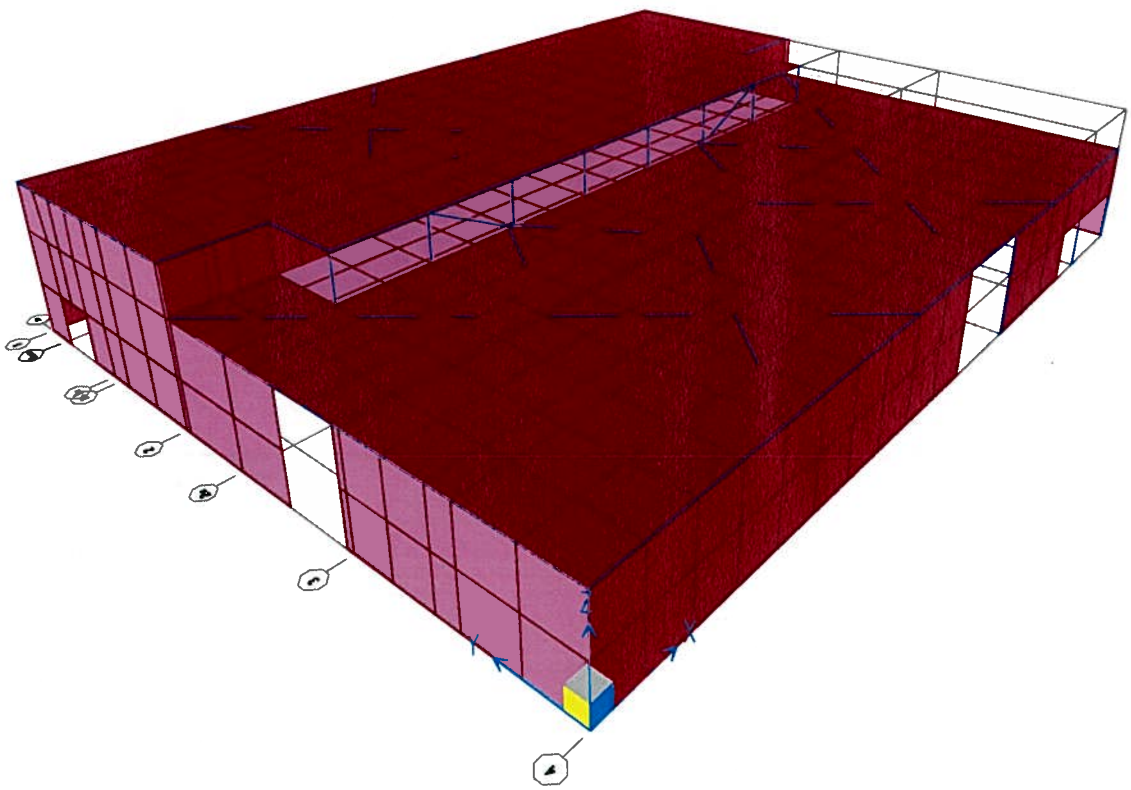
The "Increment 3" is a small rectangular one-story building with a size of 39' x 76'x 12.5'. It is attached to the north end of the main building at its longitudinal side. The structure consists of a fairly large concrete core around and a wood framed structure with a flat roof. There are no existing drawings for the "Increment 2" structure available.

B. Available existing design data:

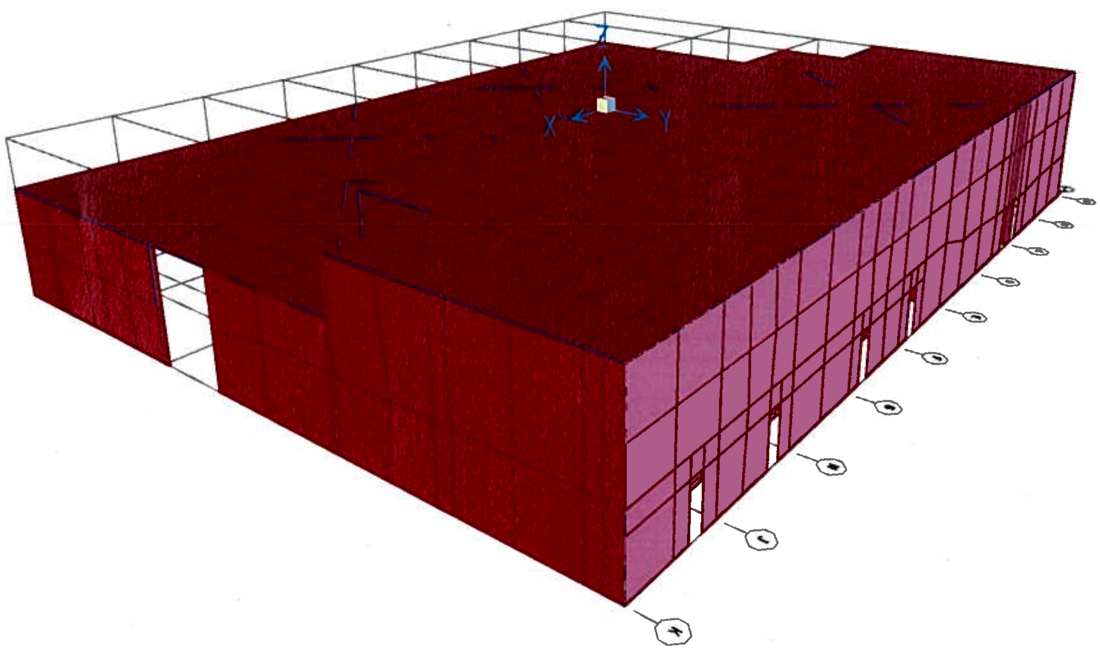
1. As built drawings for the main building and the "Increment 3" structure.
2. Design specification for main building only.
3. Structural calculations for the main building "Increment 2" and "Increment 3".

4. Design criteria :
   a. Design code:
      UBC 1961 - main building
      UBC 1973 - "Increment 3"
   b. Seismic design level (for main building):
      0.1W - structural systems
      0.2W - structural components
   c. Steel frame - A36 steel (fy = 36,000 psi)
   d. Concrete strength:
      fc' = 4,000 psi - Precast panels
      fc' = 3,000 psi - Loft and Mezzanine floor slabs
      fc' = 2,500 psi - Walls and foundations
   e. Reinforcement:
      fy = 40,000 psi
   f. Light gauge steel roof deck:
      Milcor (Inland Steel), Type B, 24" wide galvanized 18 gauge deck with 3 puddle welds at ends and side seams bottom punched 24" o.c. Allowable horizontal shear = 455 lb/ft. with no increase recommended for lateral loads (F.S. = 3.0).

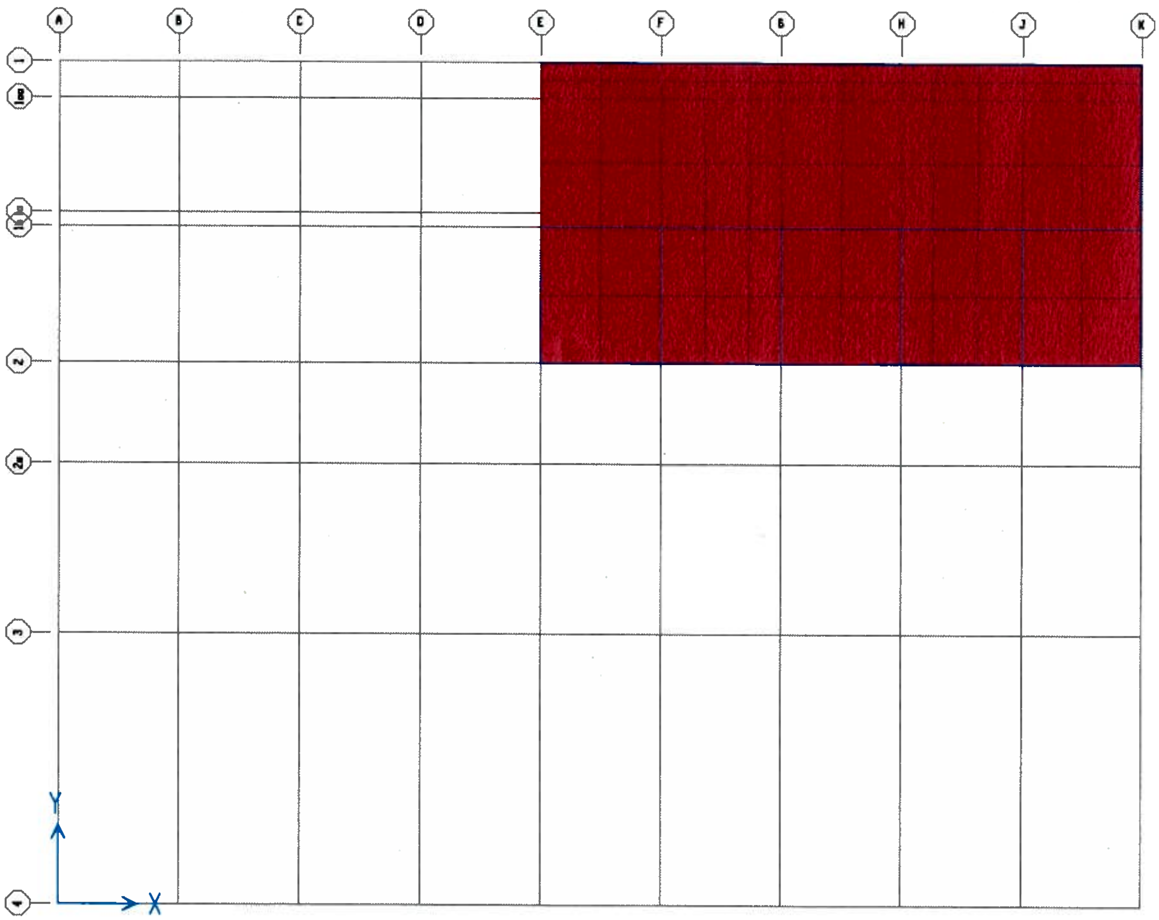

A
B
C
D
E
F
G
H
J
K
20'-0"
1
1a
1b
2
2a
3
4
25'-0"
2'-4"
22'-8"
16'-9"
28'-3"
45'-0"
HIGH ROOF ELEV. 31'-0"
LOW ROOF ELEV. 20'-0"
N
8" CONC. TO MEZZ.
8" CONC TO HIGH ROOF
6" PRE-CAST CONC.



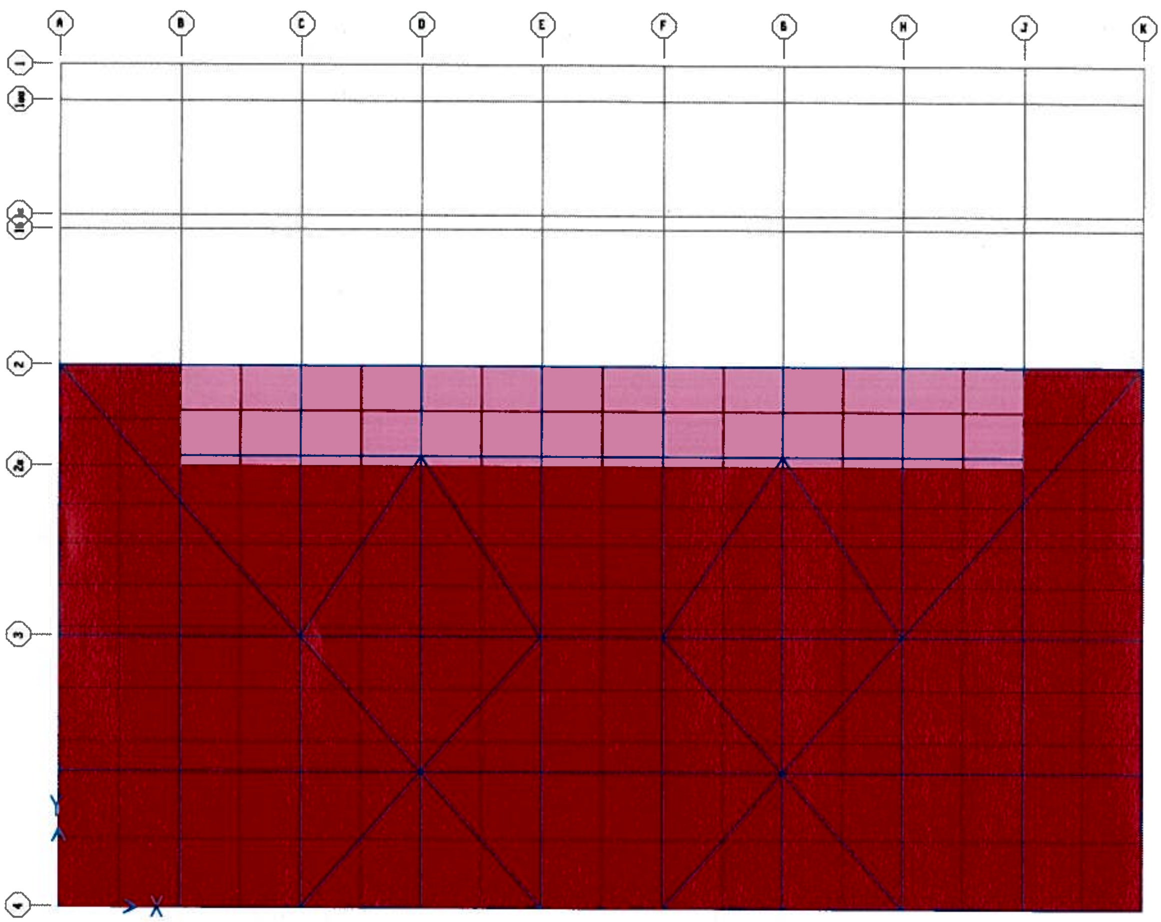
SAP2000 v15.2.1 - File:131115 B341 Increment I Model - 3-D View - Kip, in, F Units



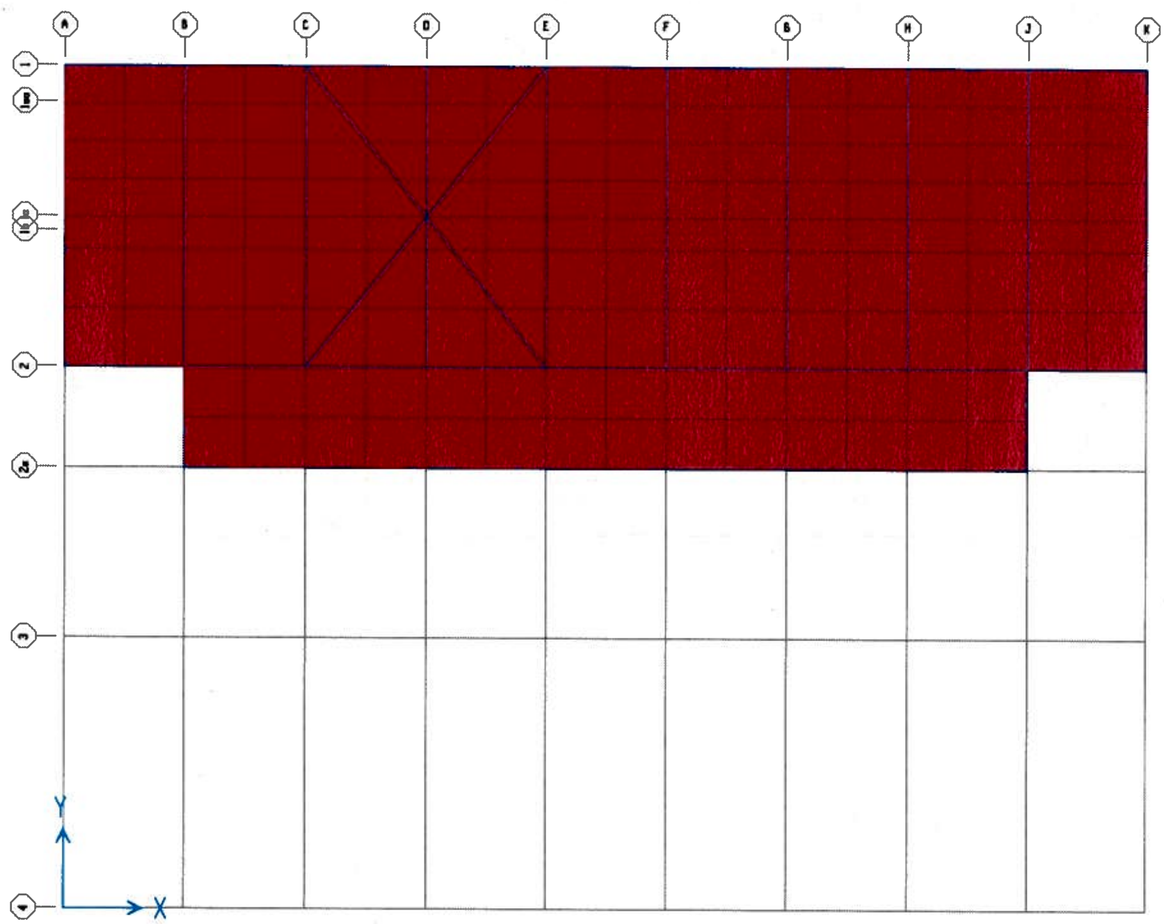
SAP2000 v15.2.1 - File:131115 B341 Increment I Model - 3-D View - Kip, in, F Units



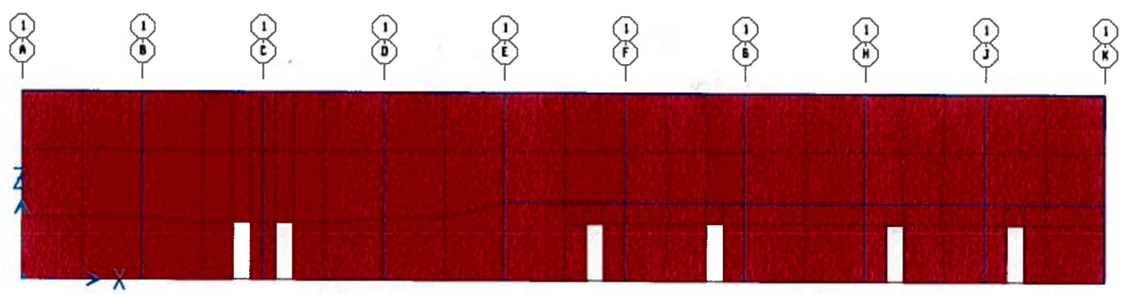
MEZZ.

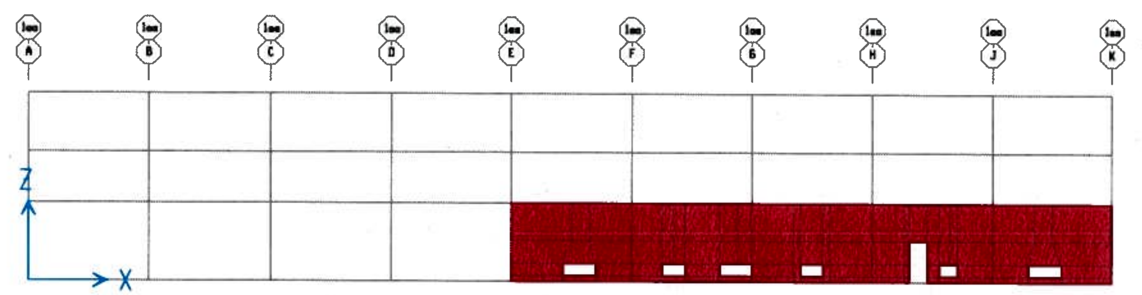


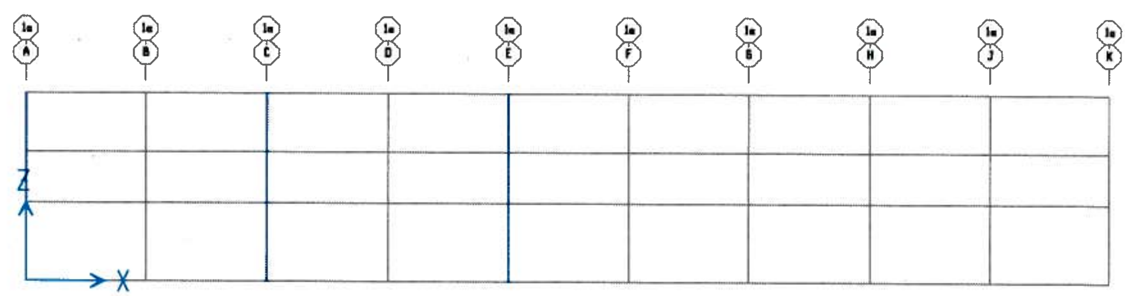
LOW ROOF / EQUIPMENT LOFT

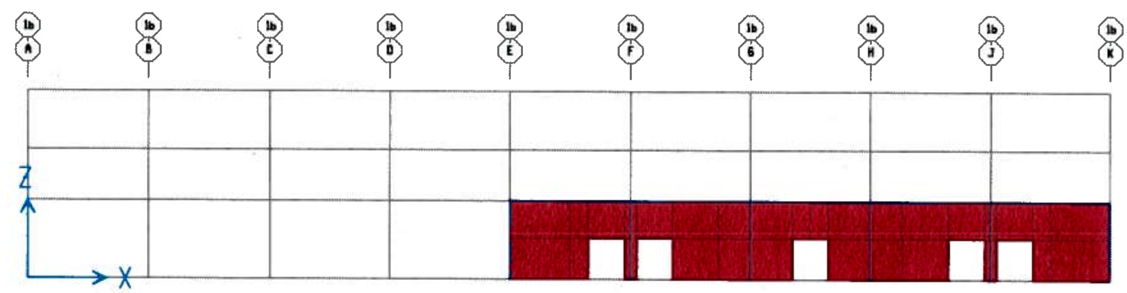


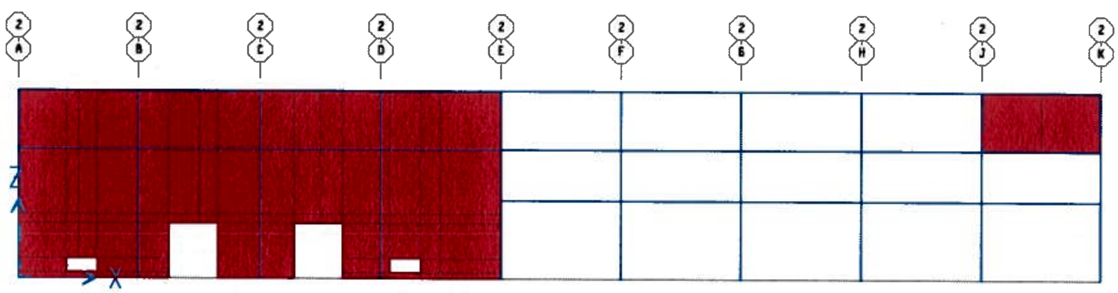
HIGH ROOF

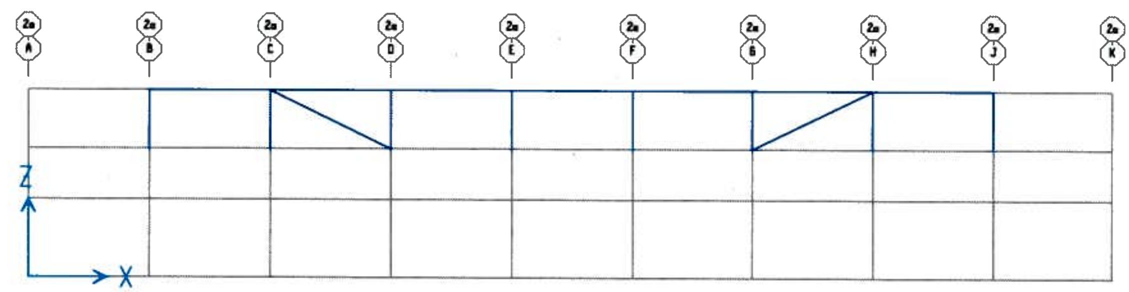

1 A
1 B
1 C
1 D
1 E
1 F
1 G
1 H
1 J
1 K
Z
X

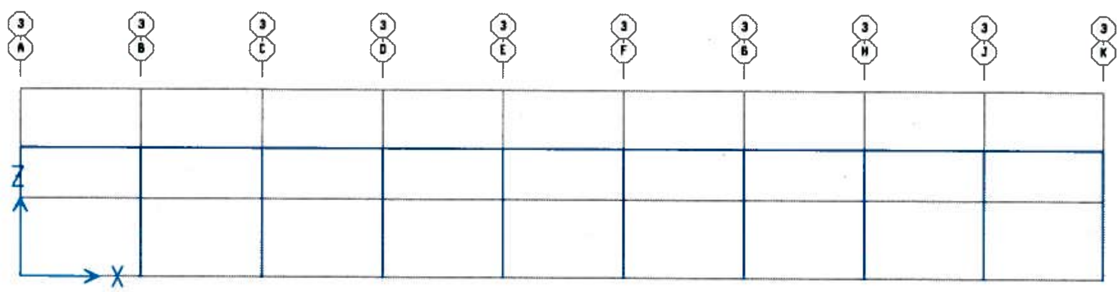

A
B
C
D
E
F
G
H
J
K
Z
X

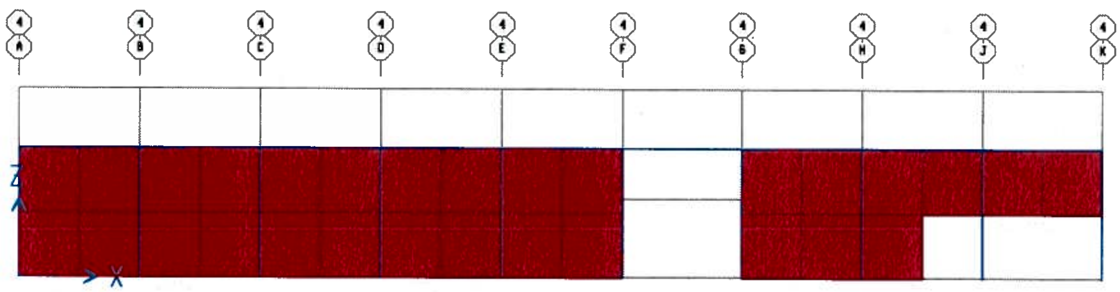

A
B
C
D
E
F
G
H
J
K
Z
X


1b
A
1b
B
1b
C
1b
D
1b
E
1b
F
1b
G
1b
H
1b
J
1b
K
Z
X

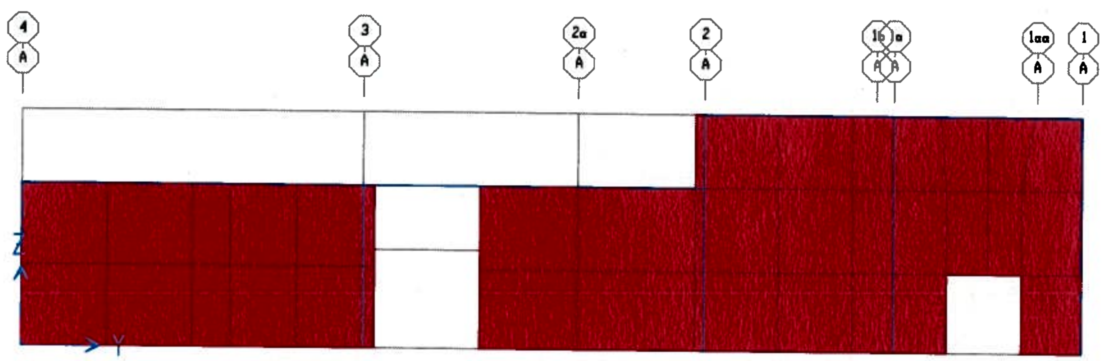

2 A
2 B
2 C
2 D
2 E
2 F
2 G
2 H
2 J
2 K
Z
X

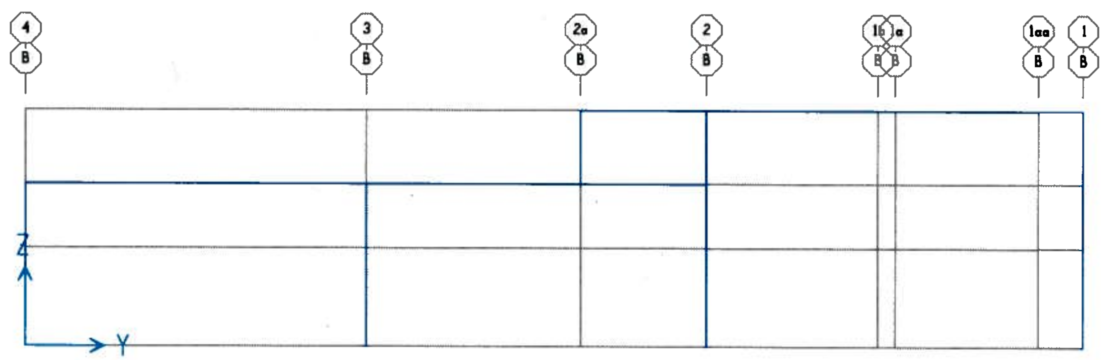

2a A
2a B
2a C
2a D
2a E
2a F
2a G
2a H
2a J
2a K
Z
X


3 A
3 B
3 C
3 D
3 E
3 F
3 G
3 H
3 J
3 K
Z
X

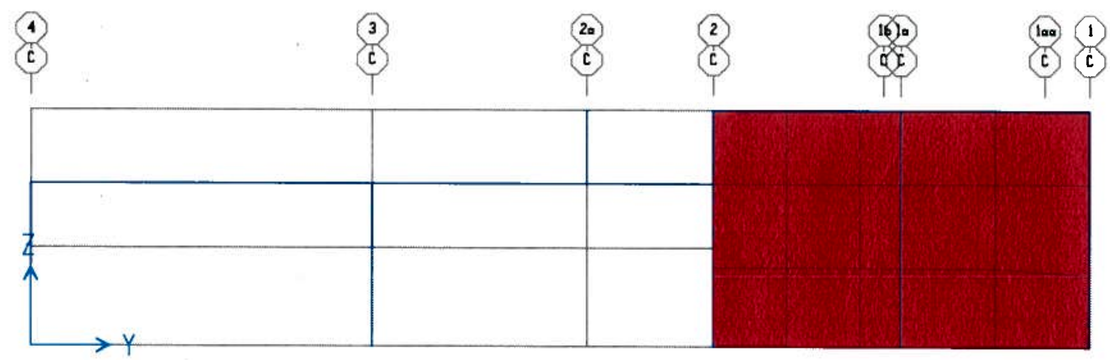


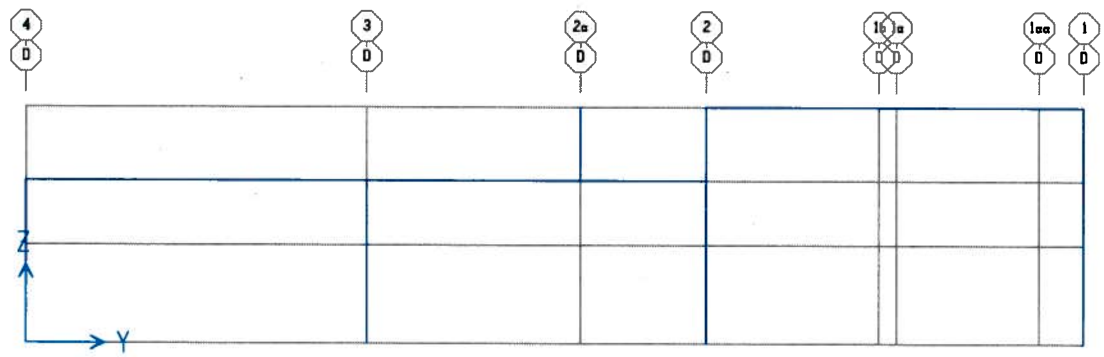
4 A
4 B
4 C
4 D
4 E
4 F
4 G
4 H
4 J
4 K
Z
X

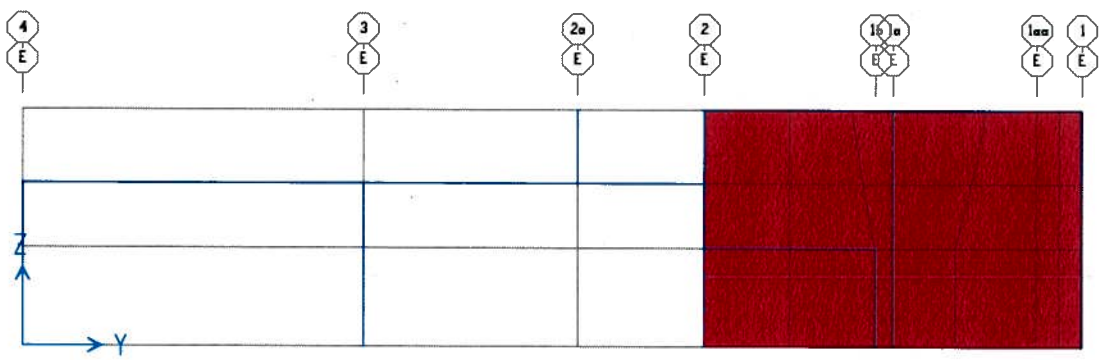

4
A
3
2a
2
1aa
1
Z
Y

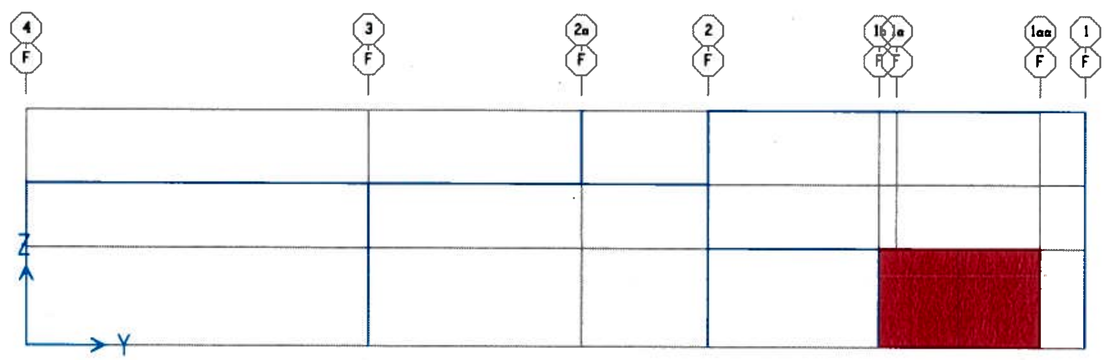


SAP2000 v15.2.1 - File:131115 B341 Increment I Model - Y-Z Plane @ X=240 - Kip, in, F Units

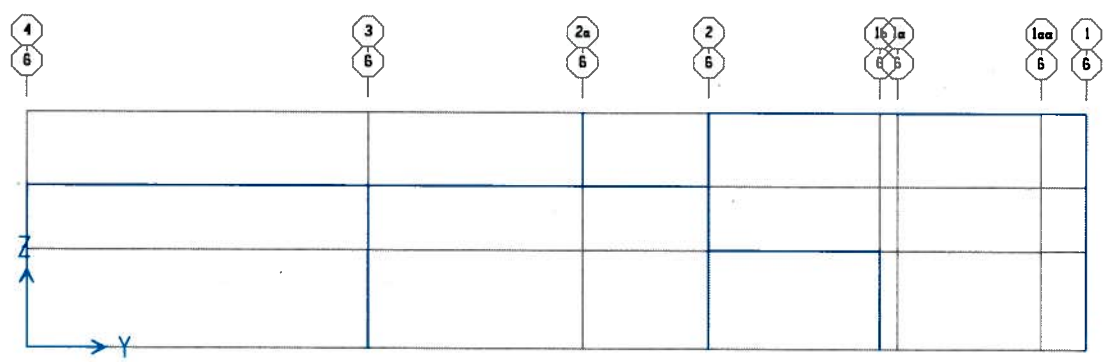

4
C
3
C
2a
C
2
C
1b
C
1a
C
1aa
C
1
C
Z
Y

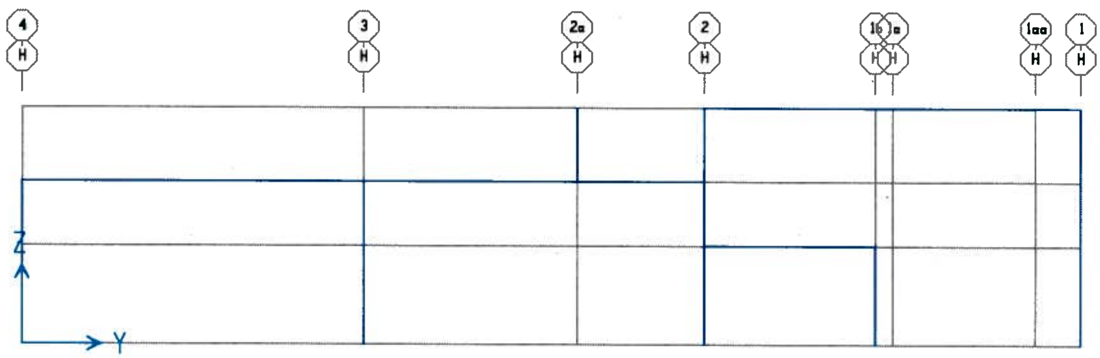

4
D
3
D
2a
D
2
D
1b
D
1a
D
1aa
D
1
D
Z
Y

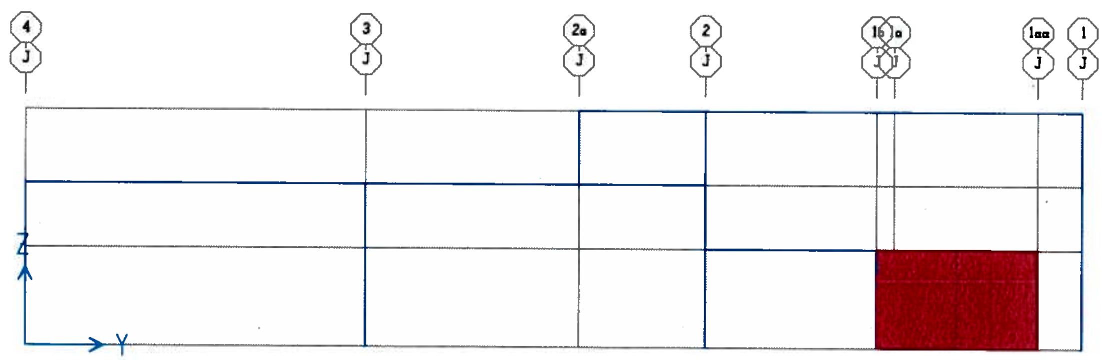

4
E
3
E
2a
E
2
E
1b
E
1a
E
1aa
E
1
E
Z
Y



SAP2000 v15.2.1 - File:131115 B341 Increment I Model - Y-Z Plane @ X=1200 - Kip, in, F Units


4
6
3
6
2a
6
2
6
1b
6
1a
6
1aa
6
1
6
Z
Y


4
H
3
H
2a
H
2
H
1b
H
1a
H
1aa
H
1
H
Z
Y


4
3
2a
2
1b
1a
1aa
1
Z
Y


4
K
3
K
2a
K
2
K
1b
K
1a
K
1aa
K
1
K
Z
Y

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject: Metal Deck Roof Diaphragm Properties	**Job Number:** B3189012.00	**Date:** 11.19.13	
Job: LLNL B341 Increment I	**By:** AMN	**Section:**	
	Checked By:		

I. General Deck Information

h :	Deck Height (in.) =	1.50	
d :	Rib Spacing (in.) =	6	
t :	Base Metal Thickness (in.) =	0.0474	18 ga, Wide Rib
w :	Panel Width (in.) =	24	
L :	Panel Length (ft.) =	30	
P :	Weld Pattern =	3	
s_e :	Edge Weld Spacing, [w/(P-1)], (in.) =	12	
s_i :	Interior Stitch Spacing (in.) =	24	
w_c:	Corrugation dimension (in.) =	1.55	
e :	Corrugation dimension (in.) =	0.875	
f :	Corrugation dimension (in.) =	3.5	

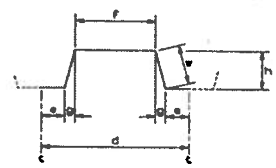


FIG 2.4-1 CORRUGATION DIMENSIONS

II. Connector Strength

Structural Fastener - Arc Spot Weld

d_w :	Weld Diameter (in.) =	0.5
F_u :	Specified Minimum Steel Strength, Members (ksi) =	45
Q_f :	Structural Connector Strength, [2.2*t*F_u*(d_w-t)] , (kips) =	2.12
S_f :	Structural Connector Flexibility, [$1.15\times10^{-3}/(t)^{0.5}$], (in./kip) =	0.0053

Sidelap Fastener - Button Punched (equivalent to #10 screw, per Verco manual)

Q_s :	Sidelap Connector Strength (kips) =	1.02	App IV
S_s :	Sidelap Connector Flexibility (in./kip) =	0.014	App IV

III. Diaphragm Strength

Edge Fasteners

α_1 :	End Dsitribution Factor, [App. IV] =	1.1
n_p :	No. of Purlins (excluding ends) =	3
α_2 :	Purlin Distribution Factor, [App. IV] =	1.1
n_e :	No. of Edge Connectors between Cross Supports, [(L/(n_p+1))/s_e] =	7
S_{ue} :	Diaphragm Strength (Edge Limit), [($2\alpha_1+n_p\alpha_2+n_e$)$Q_f$/L] (Kip/ft) =	0.88

Interior Panels

L_v :	Purlin Spacing (ft.) =	7.50
λ :	[1-h*L_v/(240*$t^{0.5}$)] =	0.78
n_s :	No. of Stitch Connectors in Length, [L/s_i] :	15
α_s :	[Q_s/Q_f] =	0.48
$\Sigma(x/w)^2$:	[App. IV] =	0.51
S_{ui} :	Diaphragm Strength (Interior Limit), [$2(\lambda-1)+n_s\alpha_s+\Sigma(x/w)^2$*($2n_p+4$)]*($Q_f$/L), (kip/ft) =	0.84

End Members

N :	No. Fasters per Foot Along Ends =	1.00
B :	[$n_s\alpha_s+\Sigma(x/w)^2$*($2n_p+4$)] =	12.29
S_u :	Diaphragm Strength (Corner Limit), [(($N^2B^2/(L^2N^2+B^2)$)$^{0.5}$)Qf], (kip/ft) =	0.81

Limiting Diaphragm Strength Value

S :	Diaphragm Strength (kip/ft) :	0.81

⇒ 810plf COMPARES WELL TO $2\times V_{ALLOW}$ = 2x455 plf = 910 plf

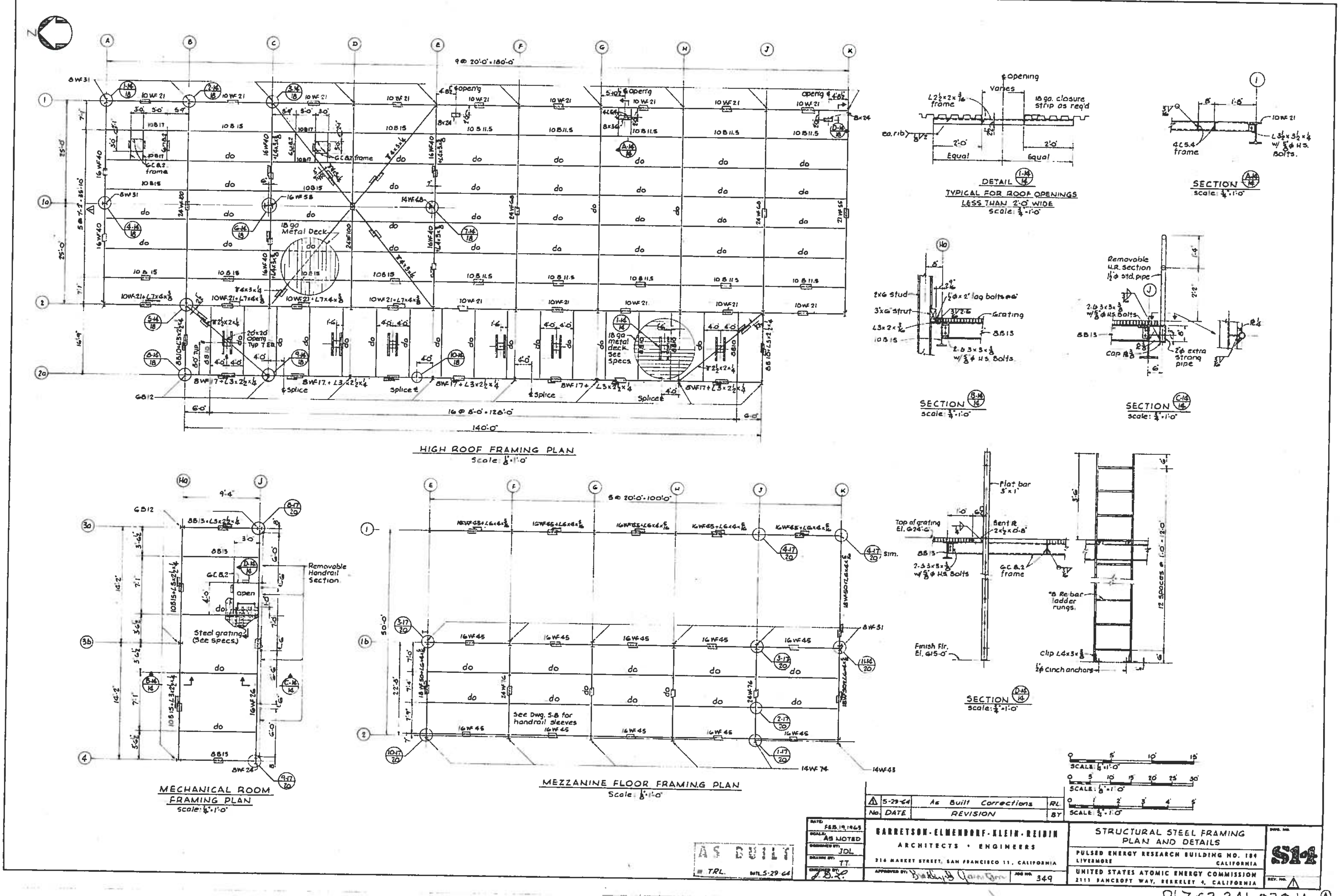
FROM 1984 EVALUATION MEMO.

IV. Stability Check

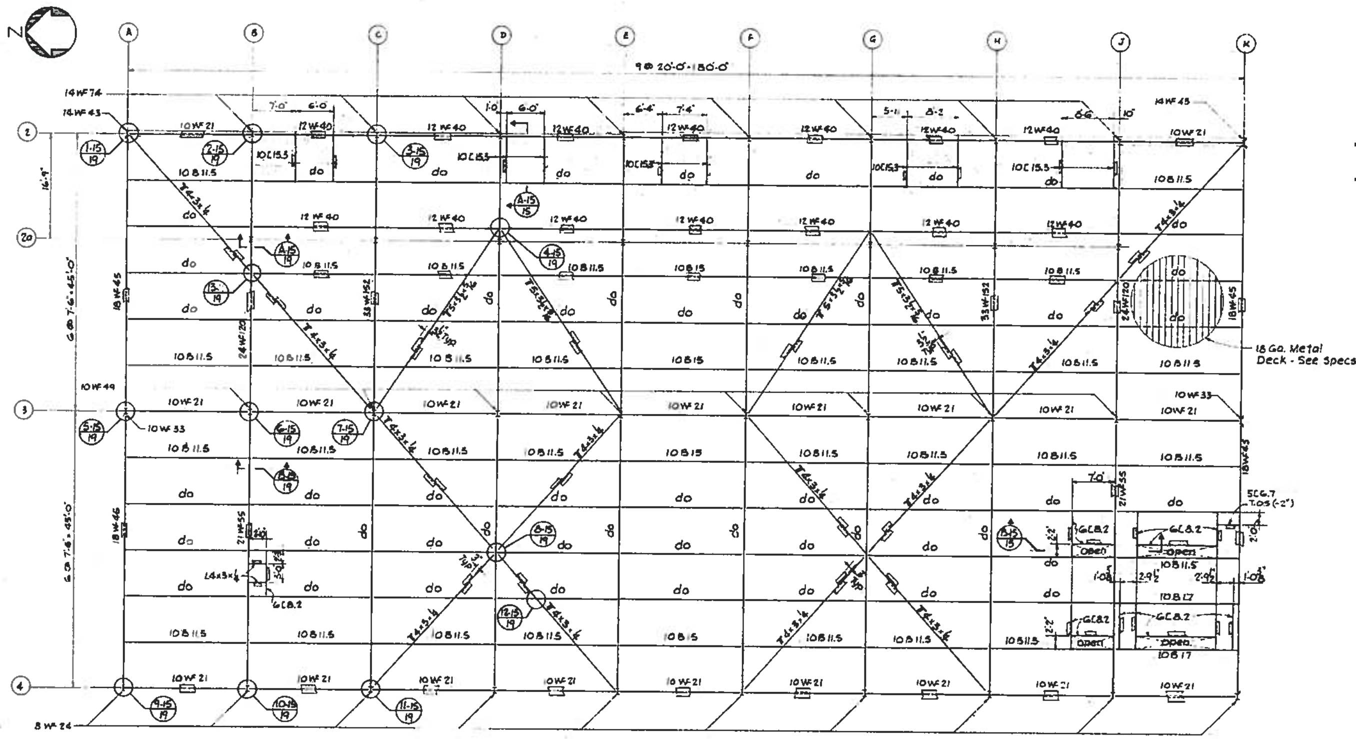
I :	Panel Moment of Inertia (in^4/ft width) =	0.305	
d_p :	Corrugation Pitch (in.) =	6	
s_d :	Developed Flute Width, [2(e+w)+f], (in.) =	8.34	1.39
S_c :	Critical Buckling Load, [($3.25\times10^3/L_v^2$)*($I^3t^3d_p/s_d$)$^{0.25}$] (kips/ft) =	2.22	

IV. Diaphragm Stiffness

E :	Modulus of Elasticity (ksi) =	29000	
υ :	Poissons Ratio =	0.3	
C :	Slip Coefficient [(Et/w)*S_f*((24L/($2\alpha_1+n_p\alpha_2+2n_sS_f/S_s$))] =	12.83	
D:	Assume Wide Rib, [Table 3.31 pg. 3-4] =	13013	18 ga, every third valley
D_n :	Warping Constant [D/12L] =	36.15	
ϕ :	D_n Reduction [Table 3.3-2] =	0.80	
G' :	Diaphragm Stiffness (kips/in) =	30.30	
G:	Effective Modulus (ksi) =	639.35	
t' :	Effective Panel Thickness [t*(G/11350)] (in.) =	0.002717	



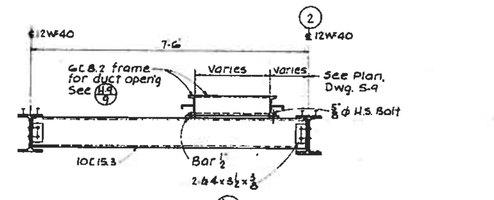
PLZ63-341-030JA



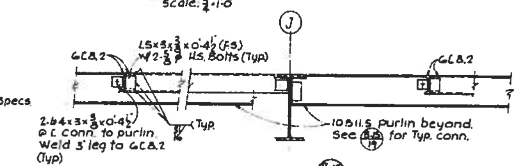
LOW ROOF FRAMING PLAN

Scale 1/8"=1'-0"

Note: Shading indicates limits of equipment loft concrete and beams having concrete anchors.



SECTION A-15/15
Scale: 3/8"=1'-0"



SECTION B-15/15
Scale: 3/4"=1'-0"

No.	DATE	REVISION	BY

AS BUILT
BY TRL DATE 5-29-64

DATE	FEB. 19 1963
SCALE	AS NOTED
DESIGNED BY	JDL
DRAWN BY	T.T.
CHECKED BY	J.B.L.

GARRETSON · ELMENDORF · KLEIN · REIBIN
ARCHITECTS · ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA

APPROVED BY: Bradley A. [illegible] JOB NO. 349

STRUCTURAL STEEL FRAMING PLAN AND DETAILS

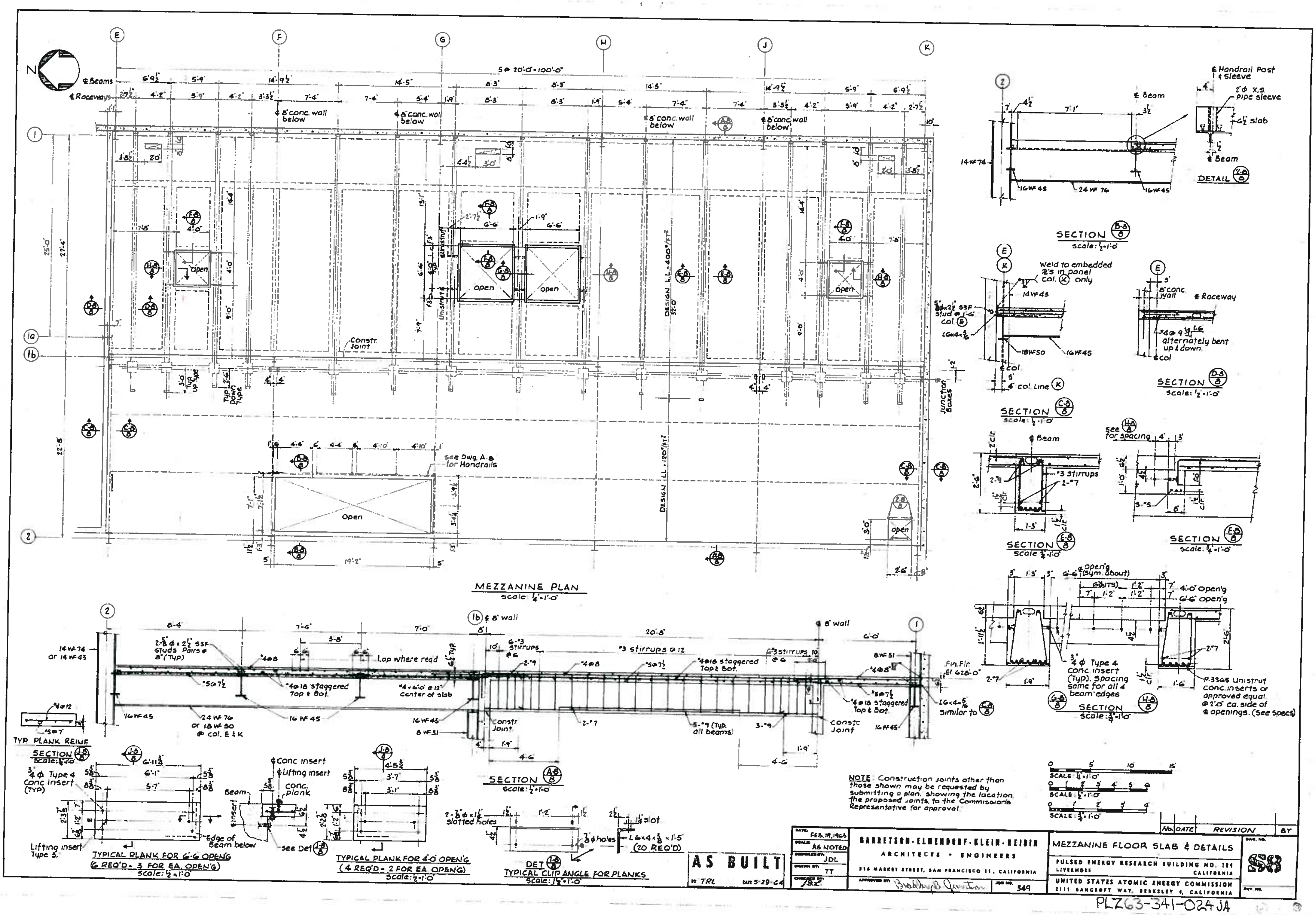
PULSED ENERGY RESEARCH BUILDING NO. 184 LIVERMORE CALIFORNIA

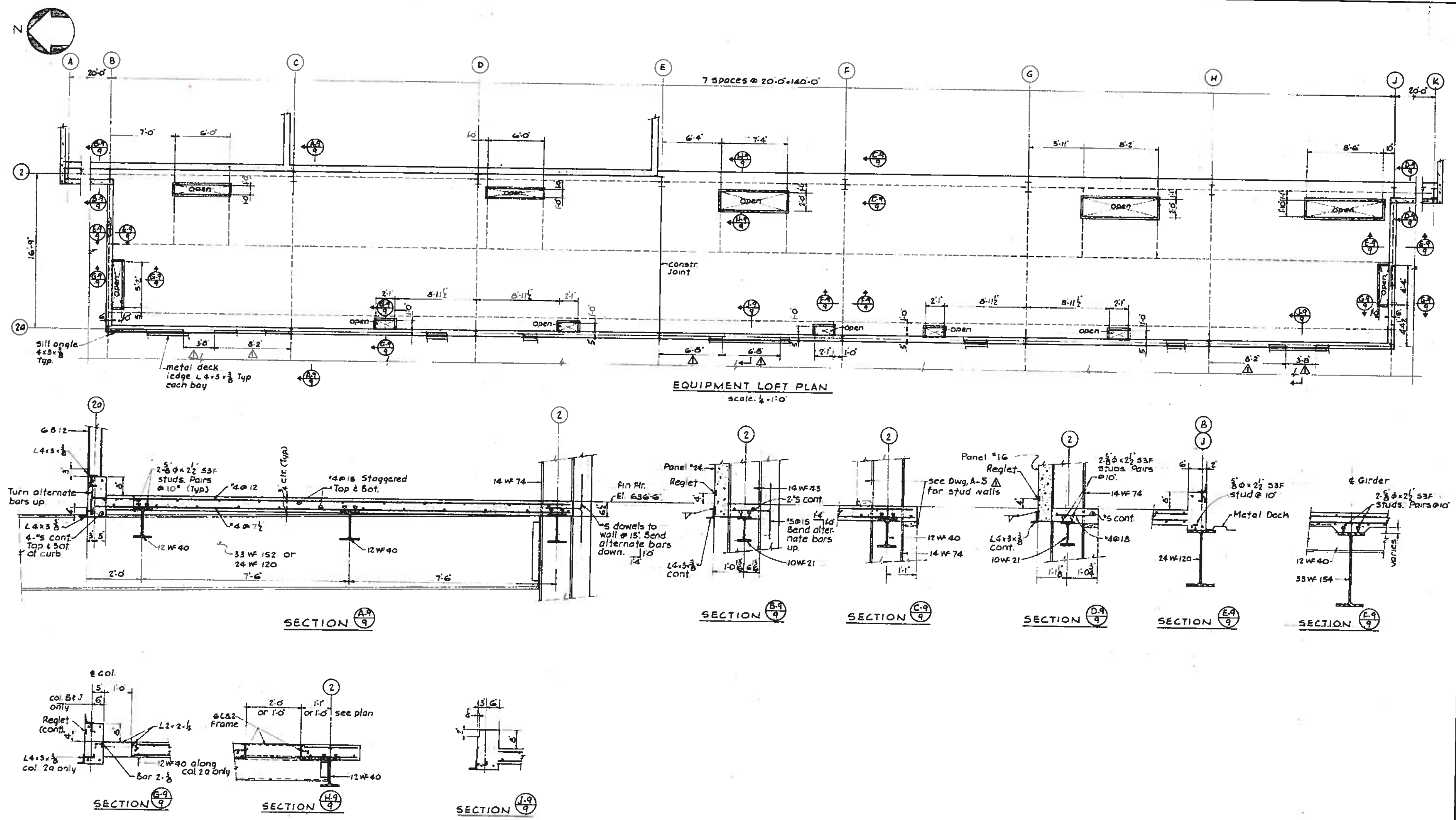
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA

DWG. NO. S15

REV. NO.

PLZ63-341-031JA


MEZZANINE PLAN
Scale: 1/4"=1'-0"
SECTION A-8
SECTION B-8
SECTION C-8
SECTION D-8
SECTION E-8
SECTION F-8
SECTION G-8 H-8
DETAIL 2-8
TYP. PLANK REINF. SECTION 1-8
TYPICAL PLANK FOR 6'-6" OPEN'G (6 REQ'D - 3 FOR EA. OPEN'G)
TYPICAL PLANK FOR 4'-0" OPEN'G (4 REQ'D - 2 FOR EA OPEN'G)
DET. 1-8 TYPICAL CLIP ANGLE FOR PLANKS
DESIGN L.L.=400#/FT²
DESIGN L.L.=120#/FT²
NOTE: Construction joints other than those shown may be requested by submitting a plan, showing the location the proposed joints, to the Commission's Representative for approval.
AS BUILT
BY TRL DATE 5-29-64
DATE FEB. 19, 1963
SCALE AS NOTED
DESIGNED BY JDL
DRAWN BY TT
BARRETSON-ELMENDORF-KLEIN-REIBIN
ARCHITECTS • ENGINEERS
316 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA
JOB NO. 349
MEZZANINE FLOOR SLAB & DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA
S8
PLZ63-341-024JA



NOTE: Construction joints other than those shown may be requested by submitting a plan, showing the location the proposed joints, to the Commission's Representative for approval.

SCALE: 1/4" = 1'-0"

SCALE: 3/4" = 1'-0"

AS BUILT

BY TRL DATE 5-29-64

No.	DATE	REVISION	BY
△	5-29-64	As built Corrections	RL

DATE: FEB 19 1963
SCALE: AS NOTED
DESIGNED BY: JDL
DRAWN BY: TT

GARRETSON · ELMENDORF · KLEIN · REIBIN
ARCHITECTS · ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA

JOB NO. 349

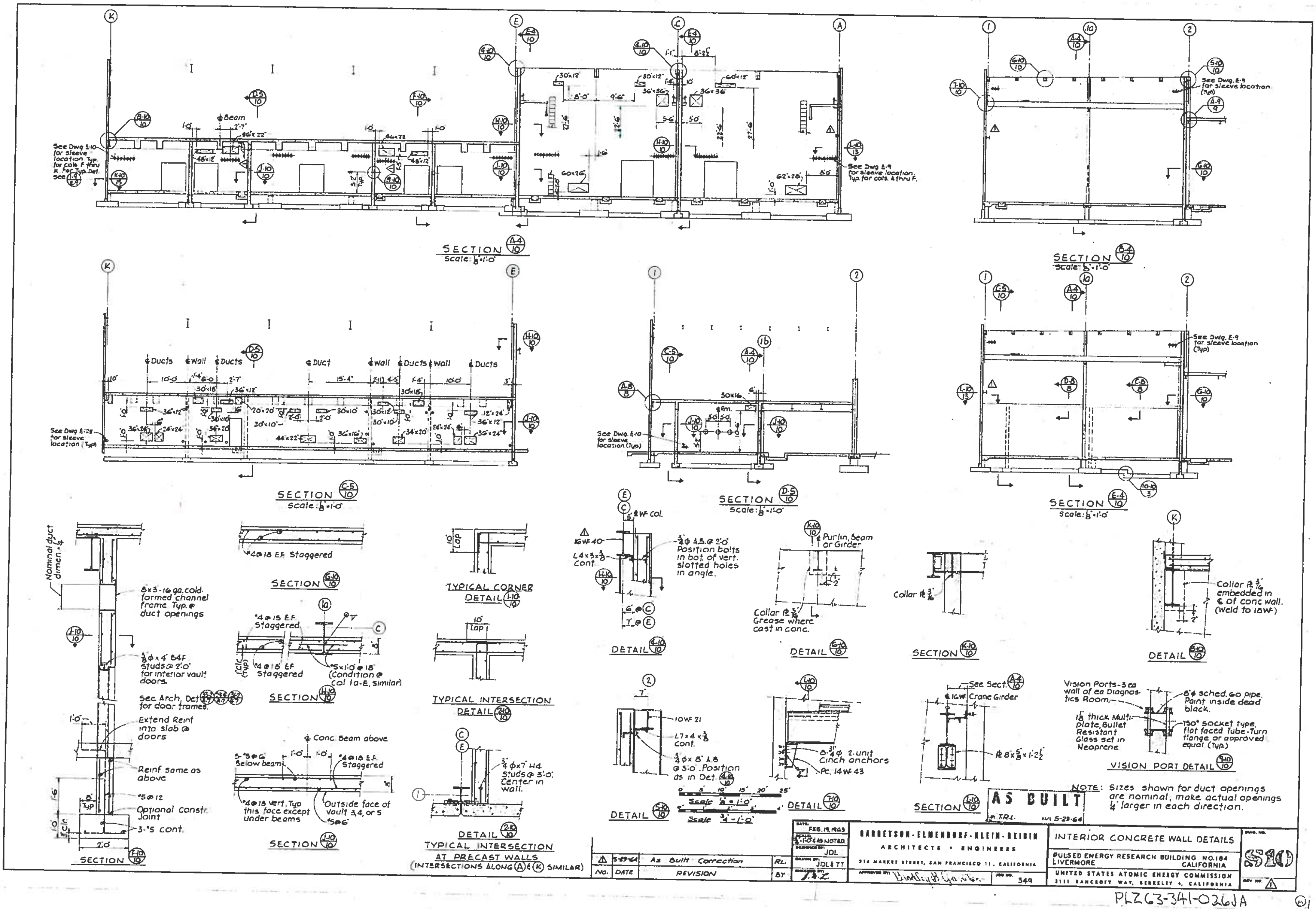
EQUIPMENT LOFT FLOOR SLAB & DETAILS

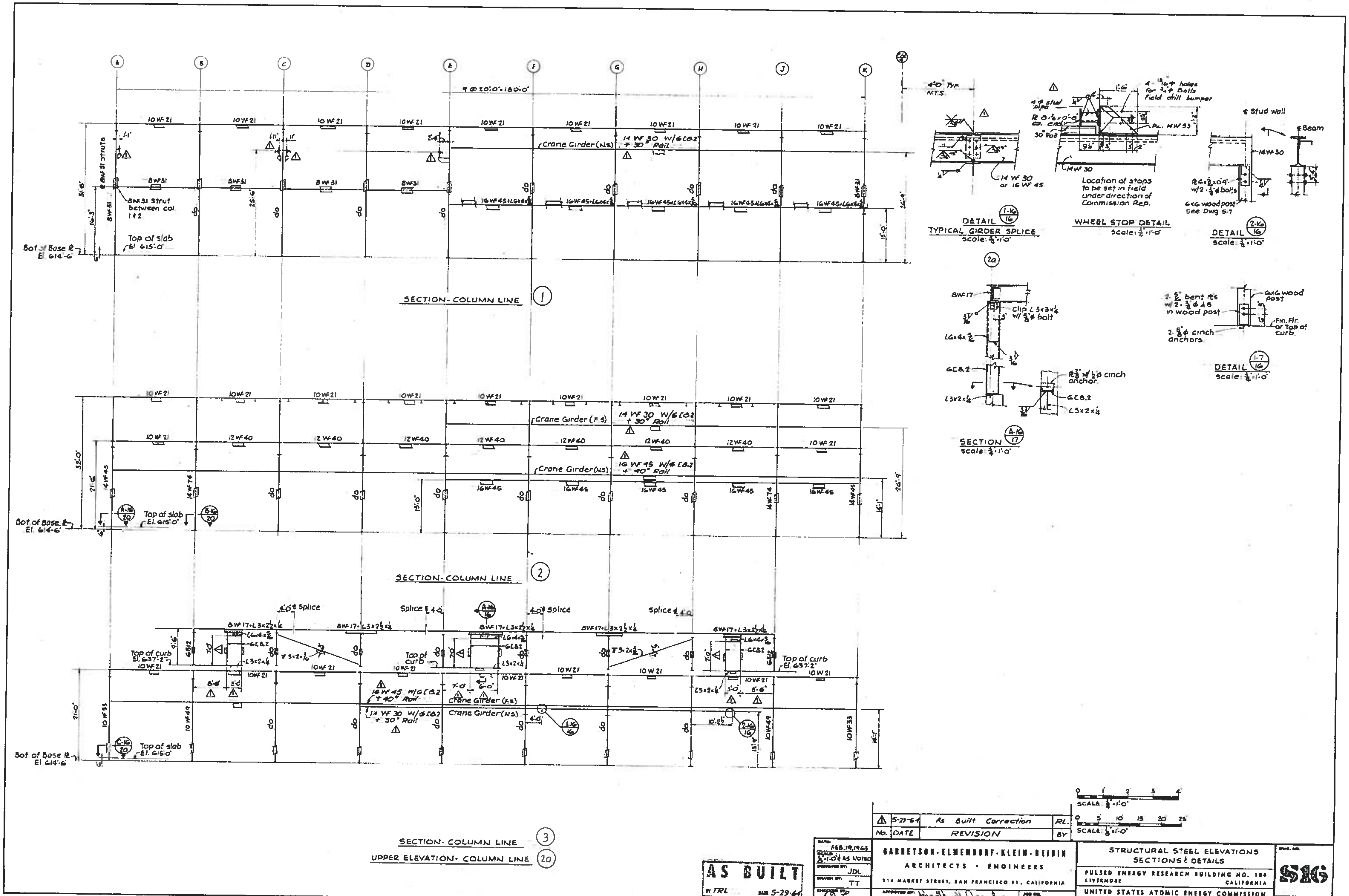
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE, CALIFORNIA

UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA

DWG. NO. S9

PLZ63-341-025JA


SECTION A-4/10
Scale: 1/8"=1'-0"
SECTION B-4/10
Scale: 1/8"=1'-0"
SECTION C-5/10
Scale: 1/8"=1'-0"
SECTION D-5/10
Scale: 1/8"=1'-0"
SECTION E-4/10
Scale: 1/8"=1'-0"
See Dwg E-10 for sleeve location Typ. for cols F thru K. For Typ. Det. See 1-8/E-4
See Dwg E-9 for sleeve location Typ. for cols. A thru F.
See Dwg. E-9 for sleeve location (Typ)
See Dwg E-25 for sleeve location (Typ)
See Dwg. E-10 for sleeve location (Typ)
Nominal duct dimen + 4"
8x3-16 ga. cold-formed channel frame Typ. @ duct openings
3/8 φ x 4" 84F Studs @ 2'-0" for interior vault doors.
See Arch. Det for door frames.
Extend Reinf into slab @ doors
Reinf same as above
#5 @ 12
Optional Constr. Joint
3-#5 cont.
SECTION F-10/10
#4 @ 18 E.F. Staggered
SECTION G-10/10
#4 @ 15 E.F. Staggered
#4 @ 18 E.F. Staggered
#5 x 1'-0" @ 18 (Condition @ Col 1a-E, similar)
SECTION H-10/10
℄ Conc. Beam above
5-#5 @ 6 Below beam
#4 @ 18 E.F. Staggered
#4 @ 18 vert. Typ this face except under beams
Outside face of vault 3,4, or 5
SECTION J-10/10
TYPICAL CORNER DETAIL 1-10/10
TYPICAL INTERSECTION DETAIL 2-10/10
3/4 φ x 7" H4 Studs @ 3'-0" Center in wall.
DETAIL 3-10/10
TYPICAL INTERSECTION AT PRECAST WALLS
(INTERSECTIONS ALONG A & K SIMILAR)
℄ W-col.
16WF40
L4x3x3/8 Cont.
3/4 φ A.B. @ 2'-0" Position bolts in bot. of vert. slotted holes in angle.
DETAIL 4-10/10
10WF21
L7x4x3/8 Cont.
3/4 φ x 8" A.B. @ 3'-0". Position as in Det. 4-10
DETAIL 5-10/10
℄ Purlin, Beam or Girder
Collar ℙ 3/16" Grease where cast in conc.
DETAIL 6-10/10
8-3/4 φ 2-unit Cinch anchors
Pc. 14WF43
DETAIL 7-10/10
Collar ℙ 3/16"
SECTION K-10/10
See Sect. A-4
℄ 16WF Crane Girder
ℙ 8 x 5/8 x 1'-2 1/2"
SECTION L-10/10
Collar ℙ 3/16" embedded in ℄ of conc wall. (weld to 18WF)
DETAIL 8-10/10
Vision Ports-3 ea wall of ea Diagnostics Room.
1 1/8 thick Multi-plate, Bullet Resistant Glass set in Neoprene
8" φ sched. 60 pipe. Paint inside dead black.
150° socket type, flat faced Tube-Turn flange or approved equal (Typ)
VISION PORT DETAIL 9-10/10
NOTE: Sizes shown for duct openings are nominal, make actual openings 1/4" larger in each direction.
AS BUILT
5-29-64
NO.	DATE	REVISION	BY
1	5-29-64	As Built Correction	RL
DATE FEB. 19, 1963
SCALE 1/8"=1'-0" & AS NOTED
DESIGNED BY JDL
DRAWN BY JDL & TT
JOB NO. 349
GARRETSON-ELMENDORF-KLEIN-REIDIN
ARCHITECTS • ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA
INTERIOR CONCRETE WALL DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA
DWG. NO. S10
PLZ63-341-026JA



SECTION- COLUMN LINE 1
SECTION- COLUMN LINE 2
SECTION- COLUMN LINE 3
UPPER ELEVATION- COLUMN LINE 2a
9 @ 20'-0" = 180'-0"
Crane Girder (N.S.)
Crane Girder (F.S.)
14 WF 30 w/6 [8.2 + 30# Rail
16 WF 45 w/6 [8.2 + 40# Rail
Top of slab El. 615'-0"
Bot of Base ℞ El. 614'-6"
Top of curb El. 637'-2"
8WF31 Strut between col 1 & 2
DETAIL 1-16/16 TYPICAL GIRDER SPLICE Scale: 3/4"=1'-0"
WHEEL STOP DETAIL Scale: 3/4"=1'-0"
Location of stops to be set in field under direction of Commission Rep.
DETAIL 2-16/16 Scale: 3/4"=1'-0"
DETAIL 1-7/16 Scale: 3/4"=1'-0"
SECTION A-16/17 Scale: 3/4"=1'-0"
AS BUILT
5-29-64 As Built Correction RL.
No. DATE REVISION BY
GARRETSON-ELMENDORF-KLEIN-REIBIN
ARCHITECTS • ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA
DATE FEB. 19, 1963
SCALE 3/4"=1'-0" & AS NOTED
DESIGNED BY JDL
DRAWN BY TT
JOB NO. 349
STRUCTURAL STEEL ELEVATIONS SECTIONS & DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184 LIVERMORE CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA
S16
PLZ63-341-032JA

Lawrence Livermore National Laboratory
Building 341 Detailed Seismic Evaluation
Structural Calculations

Evaluation of Concrete Shear Walls

December 2013

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California 94612
Phone 510.272.9040

Subject: Concrete Shear Wall Checks
Job: LLNL B341 Increment I

Job Number: B3189012 **Date:** 12.17.1
By: AMN **Section:**
Checked By:

f'c (6" walls) =	4000	psi
f'c (8" walls) =	2500	psi
fy =	40	ksi

Wall Type	Vertical Steel (in2/ft)	Horizontal Steel (in2/ft)
6" Precast	0.20	0.20
8" CIP	0.31	0.32

controlled by dowels (#5@12") into footing at 36 ksi due to incomplete development

								Shear					Flexure							Shear Friction at Base of Precast Panels		
Section Cut	Length (ft)	Height (ft)	Flange Length (ft)	Earthquake	Vu (k)	Mu (k-ft)	Wall Thickness (in)	Vu (k/ft)	sqrt's f'c	Vn (k/ft)	m-factor	DCR	As (in2)	P/Awf'c	0.9 Pdead (k)	d (ft)	Mn (k-ft)	m-factor	DCR	Vn (k/ft)	Vu/C1C2 (k/ft)	DCR
Line 1 E-K Shear Wall Mezz	100.0	32.5	0.0	BSE-1E-X	104	1027	6	1.0	0.2	19.1	2.5	0.02	20	0.01	219	48.64	53026	2.5	0.01	N/A	N/A	
Line 1 E-K Shear Wall Mezz	100.0	32.5	0.0	BSE-1E-Y	64	710	6	0.6	0.1	19.1	2.5	0.01	20	0.01	219	48.64	53026	2.5	0.01	N/A	N/A	
Line 1 Shear Wall Base	162.0	32.5	0.0	BSE-1E-X	325	5661	6	2.0	0.4	19.1	2.5	0.04	32	0.01	355	78.79	139161	2.5	0.02	2.5	1.4	0.57
Line 1 Shear Wall Base	162.0	32.5	0.0	BSE-1E-Y	56	5649	6	0.3	0.1	19.1	2.5	0.01	32	0.01	355	78.79	139161	2.5	0.02	2.5	0.2	0.10
Line 1aa Shear Wall Base	73.7	13.5	0.0	BSE-1E-X	113	2122	8	1.5	0.3	25.6	2.5	0.02	23	0.00	90	35.67	31370	2.5	0.03	11.1	1.1	0.10
Line 1aa Shear Wall Base	73.7	13.5	0.0	BSE-1E-Y	21	442	8	0.3	0.1	25.6	2.5	0.00	23	0.00	90	35.67	31370	2.5	0.01	11.1	0.2	0.02
Line 1b Sher Wall P1	13.0	13.5	0.0	BSE-1E-X	51	268	8	3.9	0.8	25.6	2.5	0.06	4	0.00	16	6.29	977	2.5	0.11	11.1	2.8	0.25
Line 1b Sher Wall P1	13.0	13.5	0.0	BSE-1E-Y	15	74	8	1.2	0.2	25.6	2.5	0.02	4	0.00	16	6.29	977	2.5	0.03	11.1	0.8	0.08
Line 1b Sher Wall P2	20.0	32.5	0.0	BSE-1E-X	86	431	8	4.3	0.9	25.6	2.5	0.07	6	0.01	59	9.68	2524	2.5	0.07	11.1	3.1	0.28
Line 1b Sher Wall P2	20.0	32.5	0.0	BSE-1E-Y	20	138	8	1.0	0.2	25.6	2.5	0.02	6	0.01	59	9.68	2524	2.5	0.02	11.1	0.7	0.06
Line 1b Sher Wall P3	20.0	22.0	0.0	BSE-1E-X	88	453	8	4.4	0.9	25.6	2.5	0.07	6	0.01	40	9.68	2407	2.5	0.08	11.1	3.2	0.28
Line 1b Sher Wall P3	20.0	22.0	0.0	BSE-1E-Y	21	128	8	1.0	0.2	25.6	2.5	0.02	6	0.01	40	9.68	2407	2.5	0.02	11.1	0.7	0.07
Line 1b Sher Wall P4	13.0	22.0	0.0	BSE-1E-X	52	306	8	4.0	0.8	25.6	2.5	0.06	4	0.01	26	6.29	1017	2.5	0.12	11.1	2.9	0.26
Line 1b Sher Wall P4	13.0	22.0	0.0	BSE-1E-Y	14	78	8	1.1	0.2	25.6	2.5	0.02	4	0.01	26	6.29	1017	2.5	0.03	11.1	0.8	0.07
Line 2 Shear Wall P1	8.0	32.5	0.0	BSE-1E-X	60	105	8	7.5	1.6	25.6	2.5	0.12	2	0.01	23	3.87	404	2.5	0.10	11.1	5.4	0.48
Line 2 Shear Wall P1	8.0	32.5	0.0	BSE-1E-Y	16	35	8	2.0	0.4	25.6	2.5	0.03	2	0.01	23	3.87	404	2.5	0.03	11.1	1.4	0.13
Line 2 Shear Wall P2	11.8	22.0	0.0	BSE-1E-X	92	282	8	7.8	1.6	25.6	2.5	0.12	4	0.01	23	5.73	843	2.5	0.13	11.1	5.6	0.50
Line 2 Shear Wall P2	11.8	22.0	0.0	BSE-1E-Y	20	40	8	1.7	0.4	25.6	2.5	0.03	4	0.01	23	5.73	843	2.5	0.02	11.1	1.2	0.11
Line 2 Shear Wall P3	12.7	32.5	0.0	BSE-1E-X	124	708	8	9.8	2.0	25.6	2.5	0.15	4	0.01	37	6.13	1011	2.5	0.28	11.1	7.0	0.63
Line 2 Shear Wall P3	12.7	32.5	0.0	BSE-1E-Y	12	69	8	0.9	0.2	25.6	2.5	0.01	4	0.01	37	6.13	1011	2.5	0.03	11.1	0.7	0.06
Line 2 Shear Wall P4	8.0	32.5	0.0	BSE-1E-X	88	244	8	11.0	2.3	25.6	2.5	0.17	2	0.01	23	3.87	404	2.5	0.24	11.1	7.9	0.71
Line 2 Shear Wall P4	8.0	32.5	0.0	BSE-1E-Y	13	33	8	1.6	0.3	25.6	2.5	0.02	2	0.01	23	3.87	404	2.5	0.03	11.1	1.1	0.10
Line 2 Shear Wall P5	13.3	13.5	0.0	BSE-1E-X	137	470	8	10.2	2.1	25.6	2.5	0.16	4	0.00	16	6.45	1027	2.5	0.18	11.1	7.3	0.66
Line 2 Shear Wall P5	13.3	13.5	0.0	BSE-1E-Y	24	143	8	1.8	0.4	25.6	2.5	0.03	4	0.00	16	6.45	1027	2.5	0.06	11.1	1.3	0.12
Line 4 A-F Shear Wall Base	100.0	13.5	0.0	BSE-1E-X	325	3492	6	3.3	0.7	19.1	2.5	0.07	20	0.00	91	48.64	50461	2.5	0.03	2.5	2.3	0.93
Line 4 A-F Shear Wall Base	100.0	13.5	0.0	BSE-1E-Y	86	1518	6	0.9	0.2	19.1	2.5	0.02	20	0.00	91	48.64	50461	2.5	0.01	2.5	0.6	0.25
Line 4 G-K Shear Wall Base	60.0	13.5	0.0	BSE-1E-X	78	1374	6	1.3	0.3	19.1	2.5	0.03	12	0.00	55	29.18	18166	2.5	0.03	2.5	0.9	0.37
Line 4 G-K Shear Wall Base	60.0	13.5	0.0	BSE-1E-Y	25	683	6	0.4	0.1	19.1	2.5	0.01	12	0.00	55	29.18	18166	2.5	0.02	2.5	0.3	0.12

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California 94612
Phone 510.272.9040

Subject: Concrete Shear Wall Checks
Job: LLNL B341 Increment I

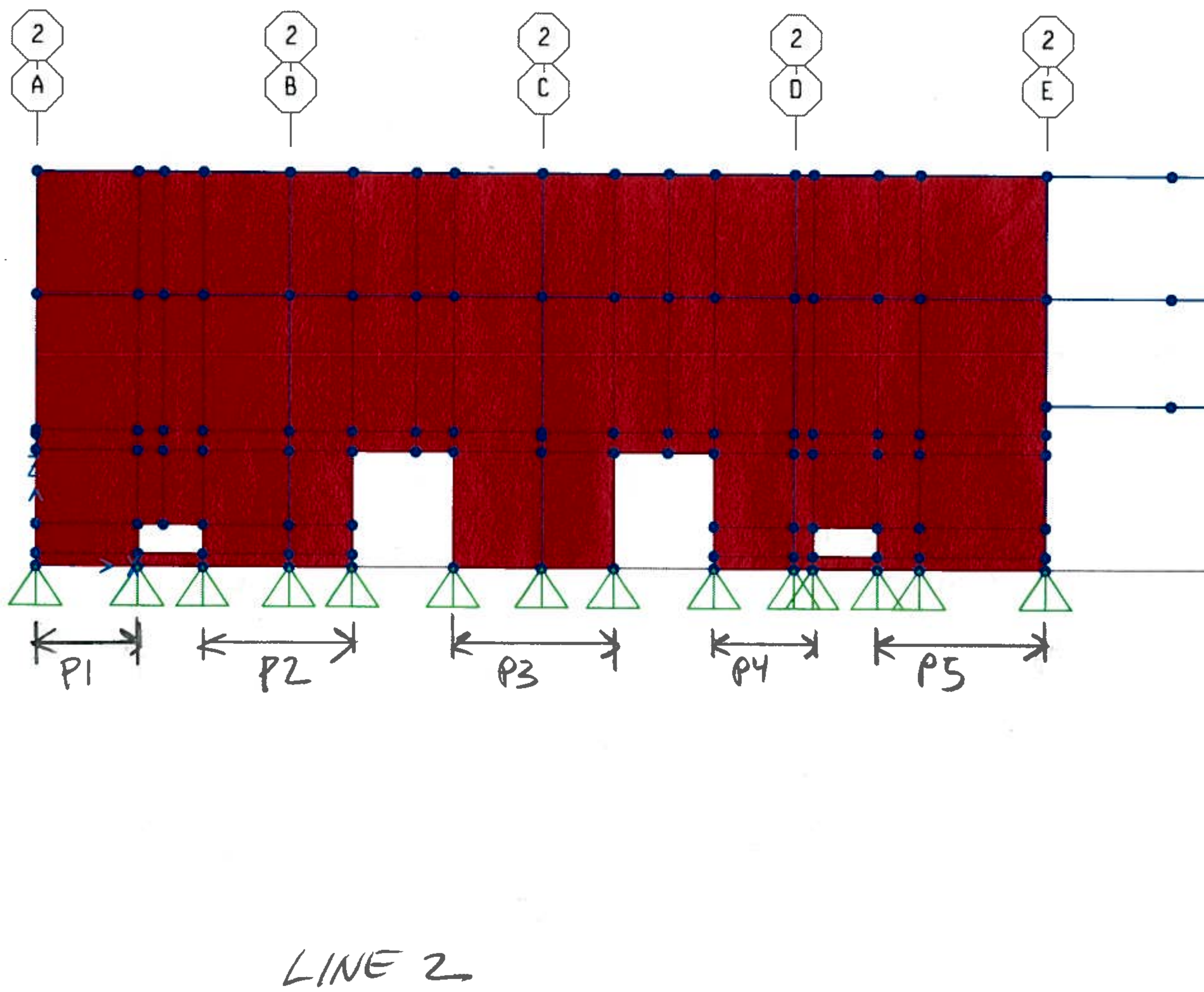
Job Number: B3189012 **Date:** 12.17.1
By: AMN **Section:**
Checked By:

f'c (6" walls) =	4000	psi
f'c (8" walls) =	2500	psi
fy =	40	ksi

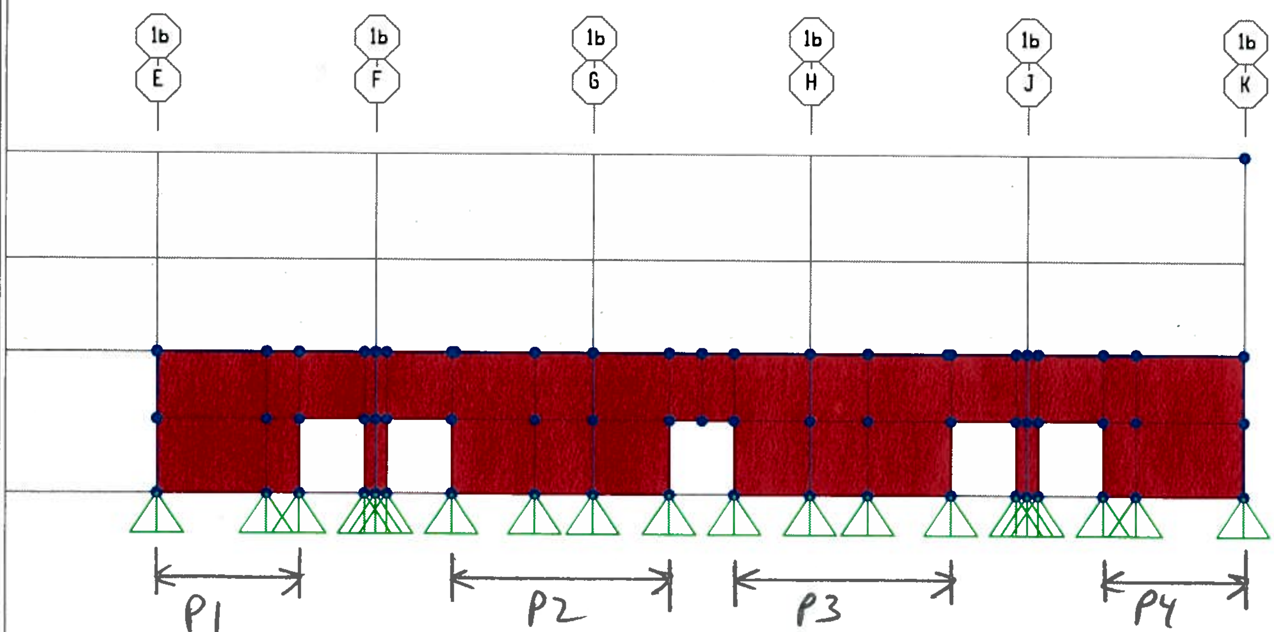
Wall Type	Vertical Steel (in2/ft)	Horizontal Steel (in2/ft)	
6" Precast	0.20	0.20	
8" CIP	0.31	0.32	controlled by dowels (#5@12") into footing at 36 ksi due to incomplete development

								Shear					Flexure							Shear Friction at Base of Precast Panels		
Section Cut	Length (ft)	Height (ft)	Flange Length (ft)	Earthquake	Vu (k)	Mu (k-ft)	Wall Thickness (in)	Vu (k/ft)	sqrt's f'c	Vn (k/ft)	m-factor	DCR	As (in2)	P/Awf'c	0.9 Pdead (k)	d (ft)	Mn (k-ft)	m-factor	DCR	Vn (k/ft)	Vu/C1C2 (k/ft)	DCR
Line A 1-2.7 Shear Wall Base	79.7	13.5	0.0	BSE-1E-X	59	1034	6	0.7	0.2	19.1	2.5	0.02	16	0.00	73	38.75	32021	2.5	0.01	2.5	0.5	0.21
Line A 1-2.7 Shear Wall Base	79.7	13.5	0.0	BSE-1E-Y	158	2088	6	2.0	0.4	19.1	2.5	0.04	16	0.00	73	38.75	32021	2.5	0.03	2.5	1.4	0.56
Line A 3-4 Shear Wall Base	46.4	32.5	0.0	BSE-1E-X	30	399	6	0.6	0.1	19.1	2.5	0.01	9	0.01	102	22.57	11416	2.5	0.01	2.5	0.5	0.18
Line A 3-4 Shear Wall Base	46.4	32.5	0.0	BSE-1E-Y	113	1310	6	2.4	0.5	19.1	2.5	0.05	9	0.01	102	22.57	11416	2.5	0.05	2.5	1.7	0.69
Line C Shear Wall Base	50.0	22.0	0.0	BSE-1E-X	55	501	8	1.1	0.2	25.6	2.5	0.02	16	0.01	99	24.21	15043	2.5	0.01	11.1	0.8	0.07
Line C Shear Wall Base	50.0	22.0	0.0	BSE-1E-Y	303	3339	8	6.1	1.3	25.6	2.5	0.09	16	0.01	99	24.21	15043	2.5	0.09	11.1	4.3	0.39
Line E Shear Wall Base	50.0	22.0	0.0	BSE-1E-X	92	3687	8	1.8	0.4	25.6	2.5	0.03	16	0.01	99	24.21	15043	2.5	0.10	11.1	1.3	0.12
Line E Shear Wall Base	50.0	22.0	0.0	BSE-1E-Y	364	5503	8	7.3	1.5	25.6	2.5	0.11	16	0.01	99	24.21	15043	2.5	0.15	11.1	5.2	0.47
Line F Shear Wall Base	21.3	22.0	0.0	BSE-1E-X	23	214	8	1.1	0.2	25.6	2.5	0.02	7	0.01	42	10.33	2738	2.5	0.03	11.1	0.8	0.07
Line F Shear Wall Base	21.3	22.0	0.0	BSE-1E-Y	116	1046	8	5.4	1.1	25.6	2.5	0.08	7	0.01	42	10.33	2738	2.5	0.15	11.1	3.9	0.35
Line F.7 Shear Wall Base	21.3	22.0	0.0	BSE-1E-X	24	196	8	1.1	0.2	25.6	2.5	0.02	7	0.01	42	10.33	2738	2.5	0.03	11.1	0.8	0.07
Line F.7 Shear Wall Base	21.3	22.0	0.0	BSE-1E-Y	134	1136	8	6.3	1.3	25.6	2.5	0.10	7	0.01	42	10.33	2738	2.5	0.17	11.1	4.5	0.40
Line H.3 Shear Wall Base	21.3	22.0	0.0	BSE-1E-X	20	181	8	0.9	0.2	25.6	2.5	0.01	7	0.01	42	10.33	2738	2.5	0.03	11.1	0.7	0.06
Line H.3 Shear Wall Base	21.3	22.0	0.0	BSE-1E-Y	132	1132	8	6.2	1.3	25.6	2.5	0.10	7	0.01	42	10.33	2738	2.5	0.17	11.1	4.4	0.40
Line J Shear Wall Base	21.3	22.0	0.0	BSE-1E-X	14	127	8	0.7	0.1	25.6	2.5	0.01	7	0.01	42	10.33	2738	2.5	0.02	11.1	0.5	0.04
Line J Shear Wall Base	21.3	22.0	0.0	BSE-1E-Y	114	1022	8	5.3	1.1	25.6	2.5	0.08	7	0.01	42	10.33	2738	2.5	0.15	11.1	3.8	0.34
Line K 1-2.7 Shear Wall Base	79.7	22.0	0.0	BSE-1E-X	75	1429	6	0.9	0.2	19.1	2.5	0.02	16	0.01	118	38.76	32782	2.5	0.02	2.5	0.7	0.27
Line K 1-2.7 Shear Wall Base	79.7	22.0	0.0	BSE-1E-Y	226	5387	6	2.8	0.6	19.1	2.5	0.06	16	0.01	118	38.76	32782	2.5	0.07	2.5	2.0	0.81
Line K 3-4 Shear Wall Base	46.4	22.0	0.0	BSE-1E-X	30	362	6	0.6	0.1	19.1	2.5	0.01	9	0.01	69	22.57	11111	2.5	0.01	2.5	0.5	0.18
Line K 3-4 Shear Wall Base	46.4	22.0	0.0	BSE-1E-Y	132	2158	6	2.9	0.6	19.1	2.5	0.06	9	0.01	69	22.57	11111	2.5	0.08	2.5	2.0	0.82
Line 2 J-K Shear Wall	20.0	10.0	0.0	BSE-1E-X	43	280	6	2.2	0.5	19.1	2.5	0.05	4	0.00	14	9.73	2000	2.5	0.06	N/A	N/A	
Line 2 J-K Shear Wall	20.0	10.0	0.0	BSE-1E-Y	55	595	6	2.7	0.6	19.1	2.5	0.06	4	0.00	14	9.73	2000	2.5	0.12	N/A	N/A	

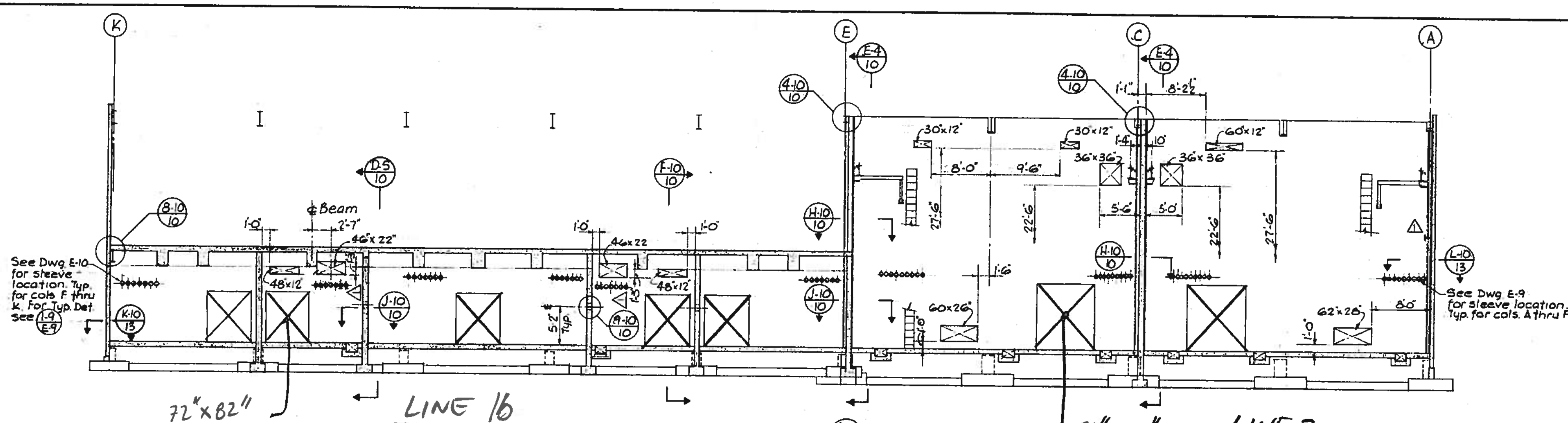
SAP2000 11/26/13 16:25:43



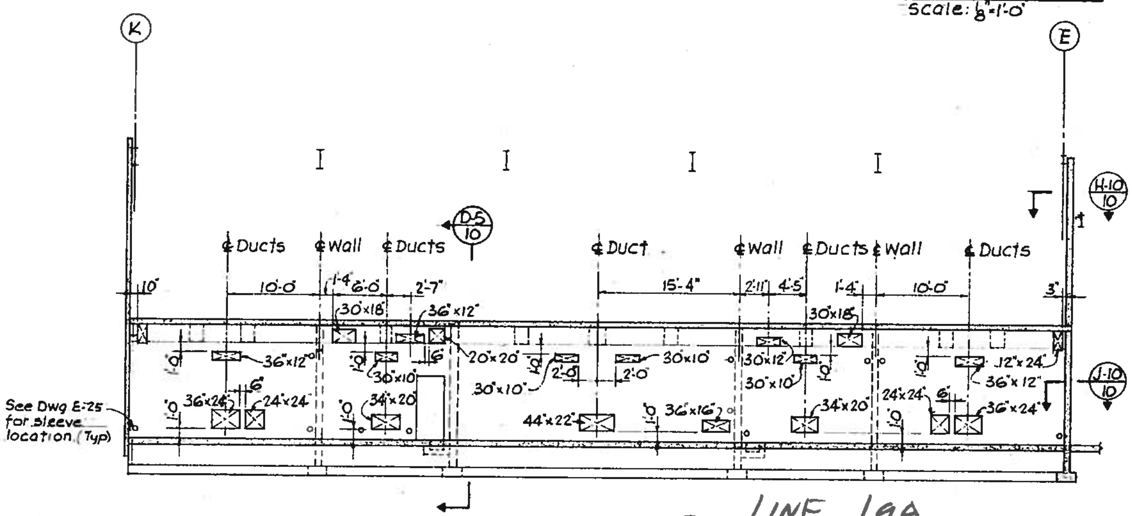
SAP2000 v15.2.1 - File:131115 B341 Increment I Model - X-Z Plane @ Y=1080 - Kip, in, F Units



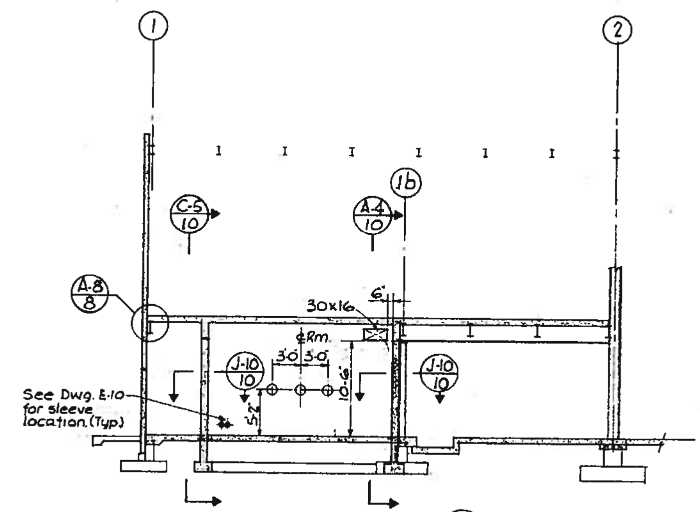
LINE 1b



SECTION A-4/10
Scale: 1/8"=1'-0"



SECTION C-5/10
Scale: 1/8"=1'-0"



SECTION D-5/10
Scale: 1/8"=1'-0"

Degenkolb

Subject: | Job Number: | Date:
Job: | By: | Section:
Checked By: | Page of

EVALUATE IN-PLANE SHEAR CONN. @ BASE OF PRECAST

SHEAR FRICTION W/ #4 @ 12" O.C. TO SLAB

$$l_{dh} = 11.2\, d_b = 5.6''$$

$$\therefore\ F_s = \frac{3''}{5.6''} \times 40^{ksi} = 21^{ksi}$$

REDUCE FOR DEVELOPMENT IN WALL

ASSUME $\mu = 0.6$ PRECAST NOT INTENTIONALL ROUGHENED

$$V_n = 0.2\,in^2 \times 21^{ksi} \times 0.6 = \underline{\underline{2.5\ ^k/ft}}$$

EVALUATE AS FORCE CONTROLLED

$$\therefore\ V_u = V_U / C_1 C_2 = V_u / 1.4$$

SHEAR FRICTION @ BASE OF CIP WALLS

#5 @ 12" O.C.

9" HOOK TO FNDN.

18" STRAIGHT DEVELOPMENT INTO WALL

$l_{dh} = 11.2\, d_b = 7'' < 9''$ OK - HOOK DEVELOPED

$l_d = 32\, d_b = 20''$ $\therefore\ F_s = \frac{18}{20} \times 40^{ksi} = 36^{ksi}$

$$V_n = 0.31\,in^2 \times 36^{ksi} \times \underbrace{1.0}_{\mu} = \underline{\underline{11\ ^k/ft}}$$

COMPARE TO 2-#4 @ 18" O.C. = 13.3 $^k/ft$
FULLY DEVELOPED

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

CHECK S.O.G. AS DIAPARAGM @ BASE OF WALLS

7" CONC SLAB w/ $\underbrace{6\times6 - 3/3}$ WWF → 0.06 in^2/ft

$F'_c = 2500$ psi

$V_{ce} = 2\sqrt{1.5 \times 2500}\,(7")(12") = 10.3^k/ft$

$V_{se} = 65^{ksi} \times 0.06\ in^2/ft \times 1.25 = 4.9^k/ft$

$V_{ne} = 10.3^k/ft + 4.9^k/ft = \underline{\underline{15.2^k/ft}}$

MAXIMUM SHEAR/FT @ PRECAST PANEL BASE

$V_u = 3.2^k/ft$ ELASTIC DEMAND

$V_{ne} = 15.2^k/ft > 3.2^k/ft$ **OK**

Degenkolb

Degenkolb Engineer
1300 Clay Street, 9th Floor
Oakland, California 94612
Phone 510.272.9040

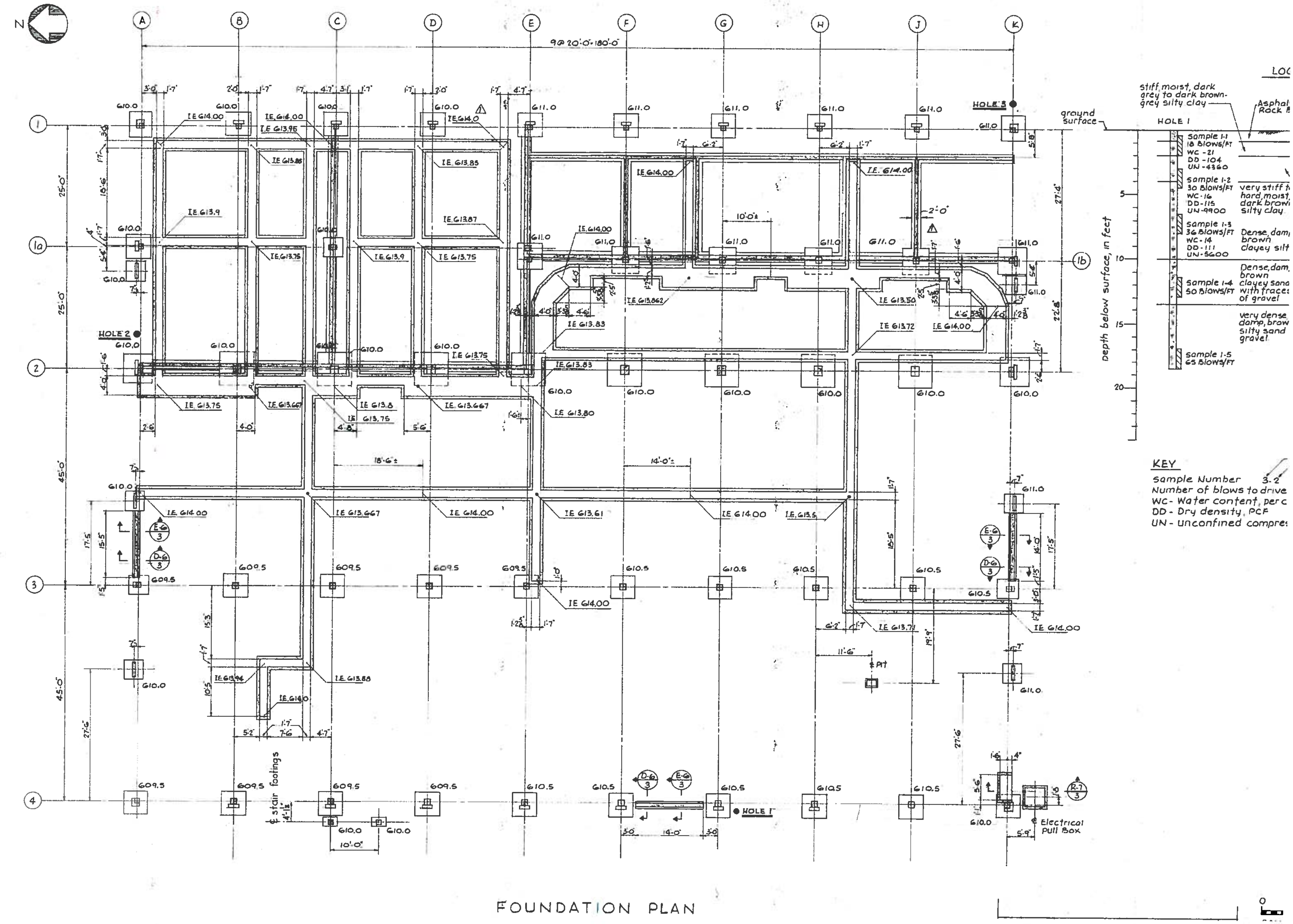
Subject: Concrete Shear Wall Over Turning Stability Checks
Job: LLNL B341 Increment I

Job Number: B3189012 **Date:** 12.17.13
By: AMN **Section:**
Checked By:

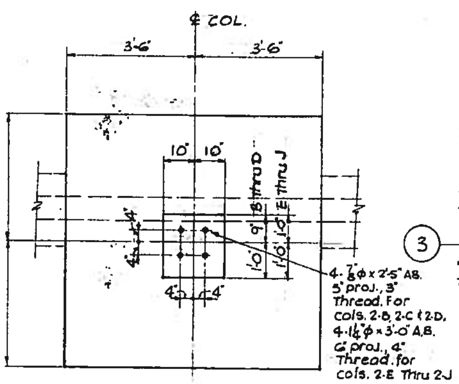
0.9Mst > Mot/C1C2μ ASCE 41-13 Equation 7-6
C1*C2 = 1.4
μ = 8 Life Safety

m-factor = 3 for soil bearing pressure check
Soil Bearing Capcity = 3*qallowable = 6 ksf

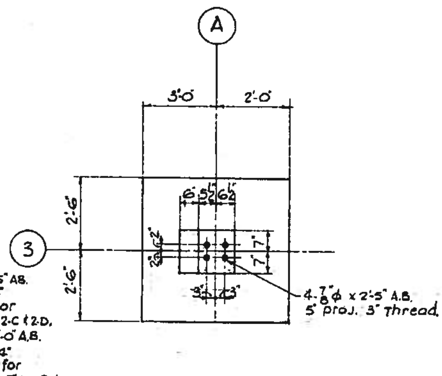
Section Cut	Length (ft)	Height (ft)	Earthquake	Mot (k-ft)	Wall Thickness (in)	Mot/(C1C2μ)	0.9 Pdead (k)	0.9 Mst (k-ft)	OT DCR	Trib Width (ft)	P trib (k)	A (sf)	S (ft^3)	P/A (ksf)	M/S/m (ksf)	Total Pressure (ksf)	DCR
Line 1 Shear Wall Base	180.0	32.5	BSE-1E-X	5661	6	505	395	35539	0.01	25	144	250	9167	2.50	0.21	2.71	0.45
Line 1 Shear Wall Base	180.0	32.5	BSE-1E-Y	5649	6	504	395	35539	0.01	25	144	250	9167	2.50	0.21	2.71	0.45
Line 2 Shear Wall Base	80.0	32.5	BSE-1E-X	9928	8	886	234	9360	0.09	66	168	284	3600	1.60	0.92	2.52	0.42
Line 2 Shear Wall Base	80.0	32.5	BSE-1E-Y	1587	8	142	234	9360	0.02	66	168	284	3600	1.60	0.15	1.75	0.29
Line 1aa Shear Wall Base	100.0	13.5	BSE-1E-X	2122	8	189	122	6075	0.03	13.75	203	200	3333	1.76	0.21	1.97	0.33
Line 1aa Shear Wall Base	100.0	13.5	BSE-1E-Y	442	8	39	122	6075	0.01	13.75	203	200	3333	1.76	0.04	1.80	0.30
Line 1b Shear Wall Base	100.0	13.5	BSE-1E-X	2140	8	191	122	6075	0.03	50	740	326	4235	2.72	0.17	2.89	0.48
Line 1b Shear Wall Base	100.0	13.5	BSE-1E-Y	502	8	45	122	6075	0.01	50	740	326	4235	2.72	0.04	2.76	0.46
Line 4 A-F Shear Wall Base	100.0	13.5	BSE-1E-X	3492	6	312	91	4556	0.07	22.5	72	150	3500	1.22	0.33	1.55	0.26
Line 4 A-F Shear Wall Base	100.0	13.5	BSE-1E-Y	1518	6	136	91	4556	0.03	22.5	72	150	3500	1.22	0.14	1.37	0.23
Line 4 G-K Shear Wall Base	60.0	13.5	BSE-1E-X	1374	6	123	55	1640	0.07	22.5	43	100	1666	1.10	0.27	1.37	0.23
Line 4 G-K Shear Wall Base	60.0	13.5	BSE-1E-Y	683	6	61	55	1640	0.04	22.5	43	100	1666	1.10	0.14	1.24	0.21
Line A 1-2.7 Shear Wall Base	79.7	13.5	BSE-1E-X	1034	6	92	73	2891	0.03	10	25	100	2281	1.14	0.15	1.29	0.22
Line A 1-2.7 Shear Wall Base	79.7	13.5	BSE-1E-Y	2088	6	186	73	2891	0.06	10	25	100	2281	1.14	0.31	1.45	0.24
Line A 3-4 Shear Wall Base	46.4	32.5	BSE-1E-X	399	6	36	102	2362	0.02	10	15	50	1125	2.78	0.12	2.90	0.48
Line A 3-4 Shear Wall Base	46.4	32.5	BSE-1E-Y	1310	6	117	102	2362	0.05	10	15	50	1125	2.78	0.39	3.17	0.53
Line C Shear Wall Base	50.0	22.0	BSE-1E-X	501	8	45	99	2475	0.02	20	148	166	2416	1.62	0.07	1.69	0.28
Line C Shear Wall Base	50.0	22.0	BSE-1E-Y	3339	8	298	99	2475	0.12	20	148	166	2416	1.62	0.46	2.08	0.35
Line E Shear Wall Base	50.0	22.0	BSE-1E-X	3687	8	329	99	2475	0.13	20	148	166	2416	1.62	0.51	2.13	0.35
Line E Shear Wall Base	50.0	22.0	BSE-1E-Y	5503	8	491	99	2475	0.20	20	148	166	2416	1.62	0.76	2.38	0.40
Line F Shear Wall Base	21.3	22.0	BSE-1E-X	214	8	19	42	450	0.04	10	32	90	664	0.92	0.11	1.03	0.17
Line F Shear Wall Base	21.3	22.0	BSE-1E-Y	1046	8	93	42	450	0.21	10	32	90	664	0.92	0.53	1.45	0.24
Line F.7 Shear Wall Base	21.3	22.0	BSE-1E-X	196	8	17	42	450	0.04	10	32	90	664	0.92	0.10	1.02	0.17
Line F.7 Shear Wall Base	21.3	22.0	BSE-1E-Y	1136	8	101	42	450	0.23	10	32	90	664	0.92	0.57	1.49	0.25
Line H.3 Shear Wall Base	21.3	22.0	BSE-1E-X	181	8	16	42	450	0.04	10	32	90	664	0.92	0.09	1.02	0.17
Line H.3 Shear Wall Base	21.3	22.0	BSE-1E-Y	1132	8	101	42	450	0.22	10	32	90	664	0.92	0.57	1.49	0.25
Line J Shear Wall Base	21.3	22.0	BSE-1E-X	127	8	11	42	450	0.03	10	32	90	664	0.92	0.06	0.99	0.16
Line J Shear Wall Base	21.3	22.0	BSE-1E-Y	1022	8	91	42	450	0.20	10	32	90	664	0.92	0.51	1.44	0.24
Line K 1-2.7 Shear Wall Base	79.7	22.0	BSE-1E-X	1429	6	128	118	4716	0.03	10	25	100	2281	1.70	0.21	1.91	0.32
Line K 1-2.7 Shear Wall Base	79.7	22.0	BSE-1E-Y	5387	6	481	118	4716	0.10	10	25	100	2281	1.70	0.79	2.49	0.41
Line K 3-4 Shear Wall Base	46.4	22.0	BSE-1E-X	362	6	32	69	1599	0.02	10	15	83	1125	1.19	0.11	1.30	0.22
Line K 3-4 Shear Wall Base	46.4	22.0	BSE-1E-Y	2158	6	193	69	1599	0.12	10	15	83	1125	1.19	0.64	1.83	0.31



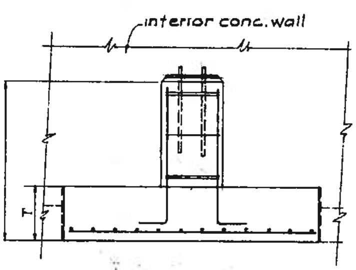
FOUNDATION PLAN



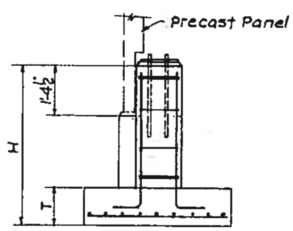
PLAN



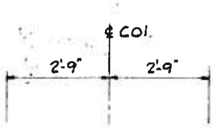
PLAN

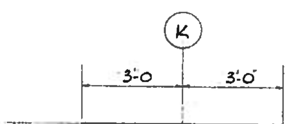


COL FTG. 2-B THRU 2-J



COL FTG 3-A & 3-K (OPP. HAND)





FOUNDATION SCHEDULE

FOOTING				PEDESTAL			
COL. MARK	SIZE N-S E-W	"T" IN.	REINFORCING	SIZE N-S E-W	VERT. REINF.	"H" FT-IN	TOP OF PED EL.
1-A	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 12"	4-#5	4'-3½"	614'-3½"
1-B	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	4'-3½"	
1-C	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	4'-3½"	
1-D	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	4'-3½"	
1-E	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-3½"	
1-F	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-3½"	
1-G	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-3½"	
1 H	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-3½"	
1-J	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-3½"	
1-K	5'-0" x 5'-0"	12"	10-#5 E.W	12" x 12"	4-#5	3'-3½"	
1a-A	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 14"	4-#5	4'-3½"	
1a-C	4'-0" x 4'-0"	12"	5-#5 E.W.	20" x 12"	4-#6	4'-3½"	
1a-E	5'-0" x 5'-6"	14	11-#5 E.W	18" x 14"	4-#6	3'-3½"	
1b-E	5'-0" x 5'-6	14"	11-#5 E.W	14" x 13"	4-#6	3'-3½"	
1b-F	5'-6" x 5'-6"	14"	11-#5 E.W.	14" x 13"	4-#6	3'-3½"	
1b-G	5'-6" x 5'-6"	14"	11-#5 E.W.	14" x 13"	4-#6	3'-3½"	
1b-H	5'-6" x 5'-6"	14"	11-#5 E.W.	14" x 13"	4-#6	3'-3½"	
1b-J	5'-6" x 5'-6"	14"	11-#5 E.W.	14" x 13"	4-#6	3'-3½"	
1b-K	6'-0" x 5'-6"	14"	11-#5 E.W.	12" x 13"	4-#6	3'-3½"	
2-A	6'-0" x 6'-0"	18"	8-#6 E.W.	12 x 21"	4-#6	4'-3½"	
2-B	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 21"	4-#6	4'-3½"	
2-C	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 21"	4-#6	4'-3½"	
2-D	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 21"	4-#6	4'-3½"	
2-E	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 24"	4-#6	4'-3½"	
2-F	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 24"	4-#6	4'-3½"	
2-G	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 24"	4-#6	4'-3½"	
2-H	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 24"	4-#6	4'-3½"	
2-J.	7'-0" x 7'-0"	18"	12-#6 E.W.	20" x 24"	4-#6	4'-3½"	
2-K	6'-0" x 6'-0"	18"	8-#6 E.W.	12" x 21"	4-#6	4'-3½"	
3-A	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 14"	4-#6	4'-9½"	
3-B	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 14"	4-#6	4'-9½"	
3-C	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 14"	4-#6	4'-9½"	
3-D	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 14"	4-#6	4'-9½"	
3-E	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 14"	4-#6	4'-9½"	
3-F	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 14"	4-#6	3'-9½"	
3-G	5'-0" x 5-0	12"	10-#5 E.W	14" x 14"	4-#6	3'-9½"	
3-H	5'-0" x 5-0	12	10-#5 E.W	14" x 14"	4-#6	3'-9½"	
3-J	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 14"	4-#6	3'-9½"	
3-K	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 14"	4-#6	3'-9½"	
4-A	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 12"	4-#5	4'-9½"	
4-B	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	4'-9½"	
4-C	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	4'-9½"	
4-D	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	4'-9½"	
4-E	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	3'-9½"	
4-F	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	3'-9½"	
4G	5'-0" x 5'-0"	12"	10-#5 E.W.	14" x 12"	4-#5	3'-9½"	
4H	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	3'-9½"	
4J	5'-0" x 5'-0"	12"	10-#5 E.W	14" x 12"	4-#5	3'-9½"	
4K	5'-0" x 5'-0"	12"	10-#5 E.W.	12" x 12"	4-#5	4'-3½"	
Stair Footing	3'-0" x 2'-2"	12"	5-#4 E.W.	14" x 14"	4-#5	4'-8"	614'-8"

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

CHECK FOUNDATION BEARING PRESSURES

- NO FOUNDATION @ PRECAST PANELS - ONLY @ COLS
- PER ASCE 41-13 FOR FIXED BASE MODEL EVALUATE SOIL AS DEFORMATION CONTROLLED w/ $m = 3.0$ (LS)
- TYP. COL FTGS:

  LINE 1: 5'x5' LINE 2: 6'x6'

  LINE 4: 5'x5'

  LINE A: 5'x5'

  LINE K: 5'x5'
- TYP. CIP WALL STRIP FTGS - 2'-0"

FOUNDATION PROPERTIES FOR SOIL PRESSURE CHECKS

LINE 1 (180' LONG - 10 - 5'x5' FTGS)

$$A = 10 \times 5' \times 5' = 250 \text{ sf}$$

$$S = \Sigma A d^2 / 90'$$

$$= 2 \times 25 \text{ sf} \times [10^2 + 30^2 + 50^2 + 70^2 + 90^2]$$

$$= \frac{825000 \text{ ft}^4}{90'} = 9167 \text{ ft}^3$$

LINE 4 A-F 100' LONG

$$A = 6 \times 5' \times 5' = 150 \text{ sf}$$

$$S = \frac{2 \times 25 \text{ sf} \times [10^2 + 30^2 + 50^2]}{50'} = 3500 \text{ ft}^3$$

LINE 4 G-K 60' LONG

$$A = 4 \times 25 = 100 \text{ sf}$$

$$S = \frac{2 \times 25 \times [10^2 + 30^2]}{30'} = 1666 \text{ ft}^3$$

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

LINE 2 (80' LONG)

$A = 5 \times 6' \times 6' = 180\ sf + 2' \times 4 \times (20'-7') = 284\ sf$

$$S = \frac{2 \times 36sf \times [20^2 + 40^2]}{40'} = 3600\ ft^3$$

LINE 19G (100' LONG x 2' STRIP)

$A = 2' \times 100' = 200\ sf$

$$S = \frac{(100')^2 \times 2'}{6} = 3333\ ft^3$$

LINE 16 (100' LONG)

$A = 6 \times 5.5' \times 5.5' + 5 \times 2' \times (20' - 5.5') = 326\ sf$

$$S = \frac{2 \times 30.25\ sf \times [10^2 + 30^2 + 50^2]}{50'} = 4235\ ft^3$$

LINE A & K 1-2.7 (80' LONG)

$A = 4 \times 5' \times 5' = 100\ SF$

$$S = \frac{2 \times 25sf \times [15^2 + 40^2]}{40'} = 2281\ ft^3$$

LINE A & K 3-4 (45' LONG)

$A = 2 \times 5' \times 5' = 50\ sf$

$$S = \frac{2 \times 25\ sf (22.5')^2}{22.5'} = 1125\ ft^3$$

Degenkolb

Subject: | Job Number: | Date:
Job: | By: | Section:
Checked By: | Page of

LINES C & E (50' LONG)

$A = 7' \times 7' + 4' \times 4' + 5' \times 5' + 2 \times 2' \times (25' - 6') = 166 \text{ sf}$

$$S = \frac{49 \text{ sf} \times 25^2 + 25 \text{ sf} \times 25^2 + (2)\left[(2' \times 19')(12.5)^2 + \frac{(2)(19)^3}{12}\right]}{25'}$$

$= 2416 \text{ ft}^3$

LINES E, F, F.7, H.3 & J

$A = 2' \times 21.3' + 2 \times 2' \times 12' = 90 \text{ sf}$ FTG RETURN

$$S = \frac{\frac{21.3^3 \times 2'}{12} + (2)\left[2' \times 12' \times 10.7'^2\right]}{10.7'} = 664 \text{ ft}^3$$

ALLOWABLE SOIL BEARING PRESSURE

$q_{ULT} = 3\, q_{ALLOW}$

q_{ALLOW} NOT GIVEN HOWEVER

TYP. PERIMETER FTG: (LOW ROOF)

$5' \times 5' = 25 \text{ sf}$

$P_{ROOF\ DEAD} = 20' \times 22.5' \times 24 \text{ psf} = 10800\#$

$P_{WALL} = 24' \times 20' \times 75 \text{ psf} = 36000\#$

$P_{TOT} = 46800\#$

$$q_{ALLOW} = \frac{46800\#}{25 \text{ sf}} = 1872 \text{ psf}$$

SAME FOOTING AS HIGH ROOF W/ 34'-6" TALL WALLS

$$q_{ALLOW} = \frac{10800\# + 51750\#}{25 \text{ sf}} = 2502 \text{ psf}$$

CONSERVATIVELY USE $\underline{2000 \text{ psf } q_{ALLOW}}$

Degenkolb

Subject: | Job Number: | Date:
Job: | By: | Section:
| Checked By: | Page of

$5' \times 5' \times 12''$ FTG w/ 10-#5 E.W.

$$M_{ne} = (40 \times 1.25)(10 \times 0.31_{in^2})\left(9'' - \frac{40 \times 1.25 \times 10 \times 0.31}{(2)(0.85)(2.5)(60)}\right)$$
$$= 1300^{k\text{-}in}$$

TYP. PEDESTAL $= 14'' \times 12''$

$$M_u = (2.5' - 0.5')^2 (W_u)/2 = 1300^{k\text{-}in}/12''/'$$

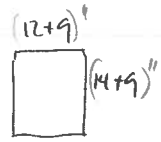
$$W_u = 54^{KSF}$$

$$V_n = (60'')(9'')(2\sqrt{2500}) = 54^k$$

$$V_u = (5' \times (2' - 9''/12''))W_u = 54^k$$
$$W_u = 8.6^{KSF}$$

PUNCHING SHEAR

$$V_c = 4\sqrt{2500}\,(\underbrace{2 \times (12'' + 9'' + 14'' + 9'')}_{b_0 = \;\; = 88''})\,9''$$



$$V_c = 158^k = (5' \times 5' - 1' \times 1.17')\,W_u$$

$$W_u = 6.6^{KSF}$$

FOOTING CAPACITY $\approx 6.6^{KSF} < \sigma_{MAX} = 3.16^{KSF}$

OK

Evaluation of Precast Concrete Panel Connections

December 2013

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject: Precast Panel Interconnection Checks	**Job Number:** B3189012.00	**Date:** 12.17.13
Job: LLNL B341 Increment I	**By:** AMN	**Section:**
	Checked By:	

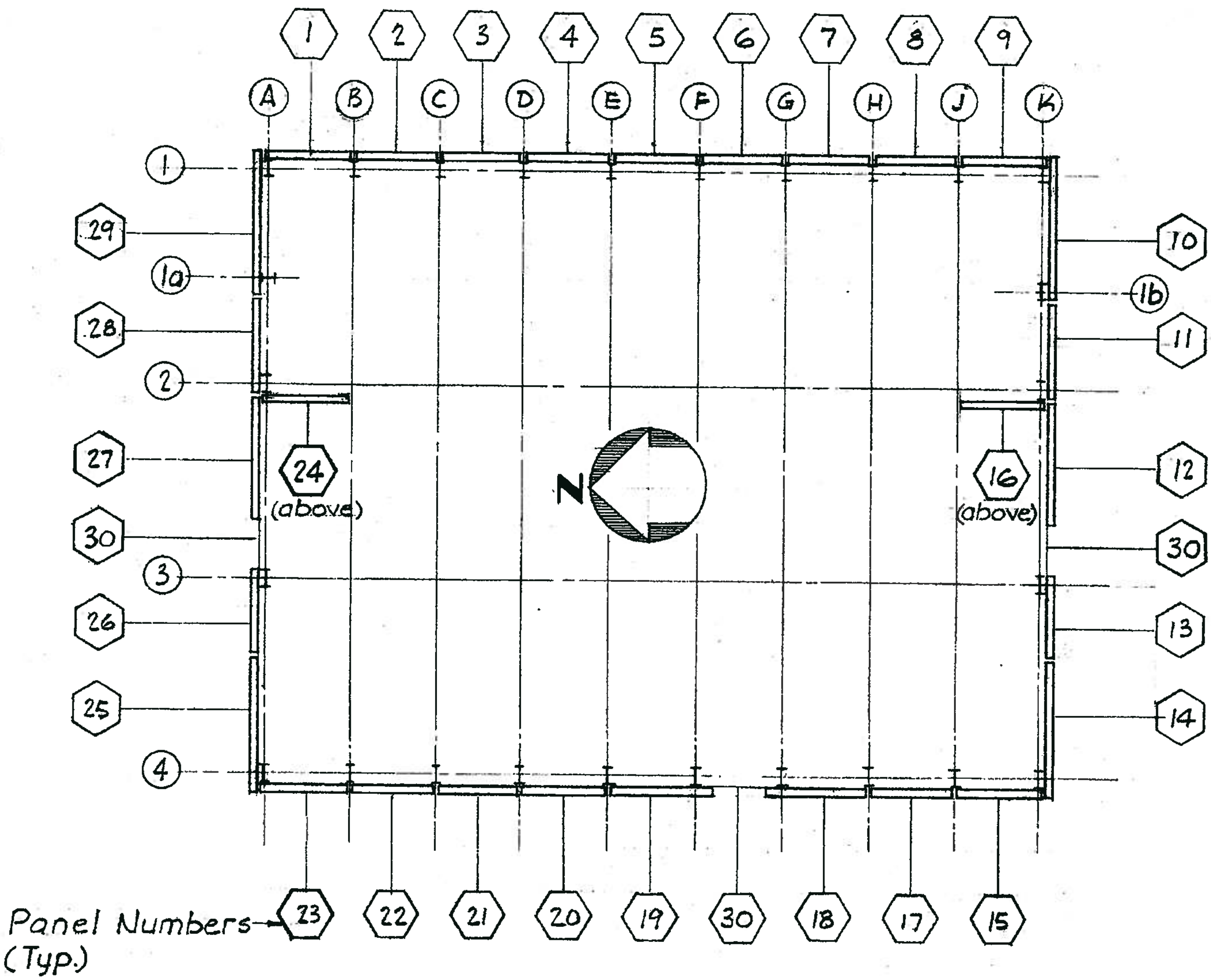
Precast Panel Connections are Force Controlled, therefore divide model demand by C1*C2 = 1.4

C1*C2 = 1.4

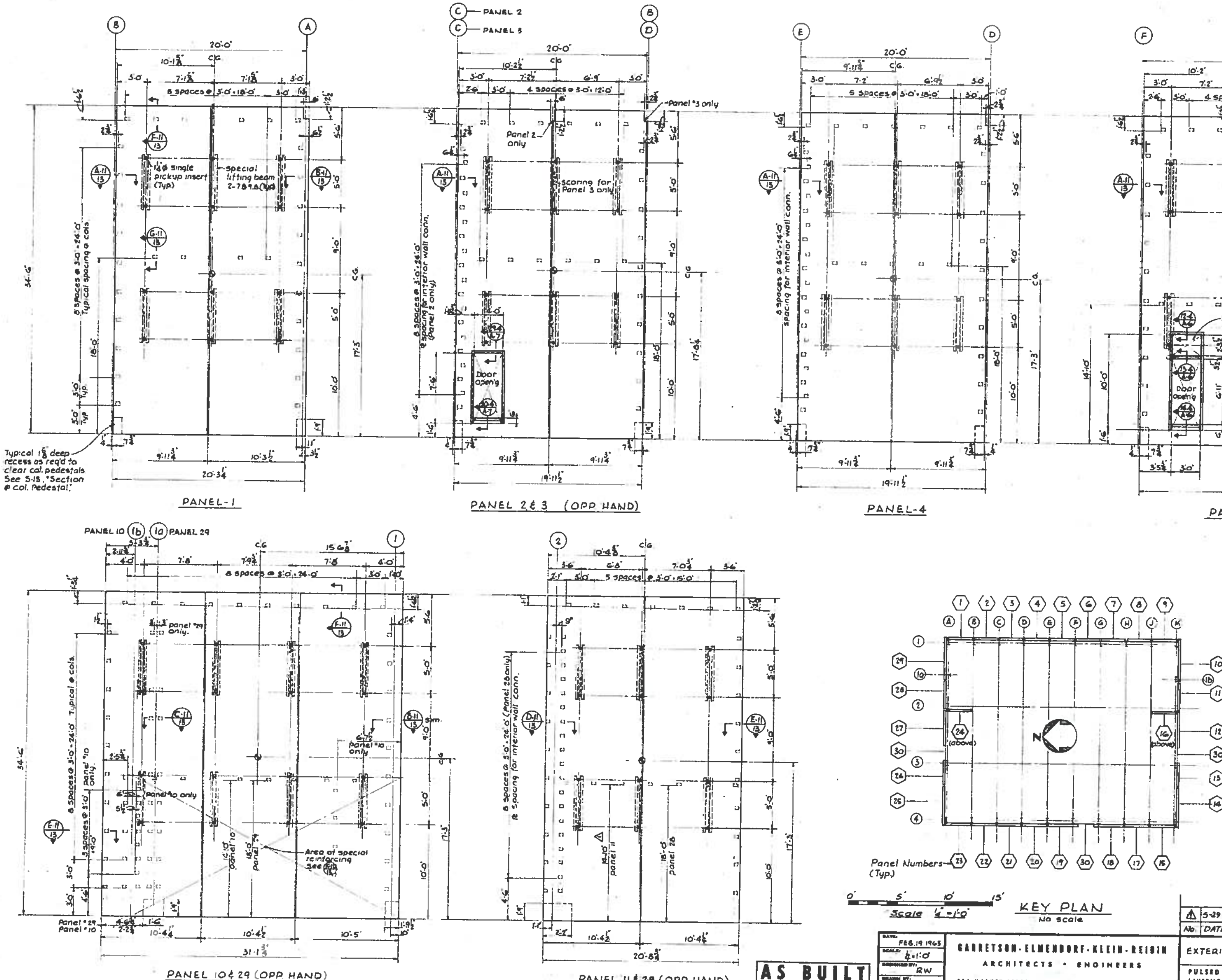
Panel-Panel Conn Capacity = 4.0 kips
Panel-WF Col Conn Capacity = 5.1 kips
Panel-WF Col @ Corner = 8.8 kips

Double Sided Col Conn Capacity = 10.0 kips

Panel Number	Panel Width (ft)	Panel Height (ft)	Horizontal Shear (k/ft)	Horiztonal Shear (k)	Vertical Shear (k)	Number of Connectors - Side 1	Connector Capacity - Side 1	DCR - Side 1	Number of Connectors - Side 2	Connector Capacity - Side 2	DCR - Side 2
1	20.0	31.6	1.4	28.7	45.3	10	8.8	0.51	10	5.1	0.89
2	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	19	5.1	0.47
3	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	5.1	0.89
4	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	19	5.1	0.47
5	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	5.1	0.89
6	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	5.1	0.89
7	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	5.1	0.89
8	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	5.1	0.89
9	20.0	31.6	1.4	28.7	45.3	10	5.1	0.89	10	8.8	0.51
10	31.2	31.6	2.0	63.2	64.1	10	8.8	0.73	10	4.0	1.60
11	20.8	31.6	2.0	42.1	64.1	10	4.0	1.60	10	4.0	1.60
12	28.5	21.3	2.0	57.8	43.1	7	4.0	1.54	7	4.0	1.54
13	18.9	21.3	2.0	38.6	43.3	7	10.0	0.62	7	4.0	1.55
14	28.3	21.3	2.0	57.8	43.3	7	4.0	1.55	7	8.8	0.70
15	20.0	7.8	0.9	18.6	7.2	4	8.8	0.20	4	5.1	0.35
17	20.0	21.3	0.9	18.6	19.7	4	5.1	0.97	7	5.1	0.55
18	20.0	21.3	0.9	18.6	19.7	7	5.1	0.55	7	10.0	0.28
19	20.0	21.3	2.3	46.5	49.4	7	10.0	0.71	7	5.1	1.38
20	20.0	21.3	2.3	46.5	49.4	7	5.1	1.38	7	5.1	1.38
21	20.0	21.3	2.3	46.5	49.4	7	5.1	1.38	7	5.1	1.38
22	20.0	21.3	2.3	46.5	49.4	7	5.1	1.38	7	5.1	1.38
23	20.0	21.3	2.3	46.5	49.4	7	5.1	1.38	7	8.8	0.80
25	28.3	21.3	1.7	49.2	36.9	7	8.8	0.60	7	4.0	1.32
26	18.9	21.3	1.7	32.8	36.9	7	4.0	1.32	7	10.0	0.53
27	28.5	21.3	1.4	40.3	30.0	7	4.0	1.07	7	4.0	1.07
28	20.8	31.6	1.4	29.4	44.6	10	4.0	1.12	10	4.0	1.12
29	31.2	31.6	1.4	44.0	44.6	10	4.0	1.12	10	8.8	0.51



of
po
4. Pa
5. Pl
on
en
6. Tw
"ins
7. Ju
th
as
8. Ea
co
ho
be
cli
9. In
sh
or
10. St
sh
tie
th
stu
11. Co
of



GENERAL NOTES

1. All elevations of panels are viewed from the outside of the building.
2. All scoring of panels is on the exterior face
3. All plate inserts are flush on the inside face of the panels and shall be held down firmly in position so as to prevent shifting or floatation.
4. Panels shall be cast with outside face up.
5. Place 2-#4x4'-0" @ 5" E.F. centered diagonally on each corner of all door openings and re-entrant corners
6. Two #4x4'-0" reinforcing bars shall be placed inside and above each double pickup insert.
7. Just prior to placing premoulded joint filler the butting edges shall be painted with asphaltic paint binder.
8. Each panel shall be securely attached to its column before the adjacent panel is erected, however the entire wall shall be in position before final tightening of bolts or welding of clip angles.
9. In case of interference, pickup insert locations shall take precedence over the other inserts or reinforcing
10. Stud bolts for column plate attachments shall be positioned first, after which the tie ℞ shall be positioned and welded with the slotted hole bearing on the top of the stud bolt
11. Contractor shall verify all sizes and locations of openings in panels.

No.	DATE	REVISION	BY
1	5-29-64	As Built Corrections	RL.

AS BUILT
BY: TRL DATE 5-29-64

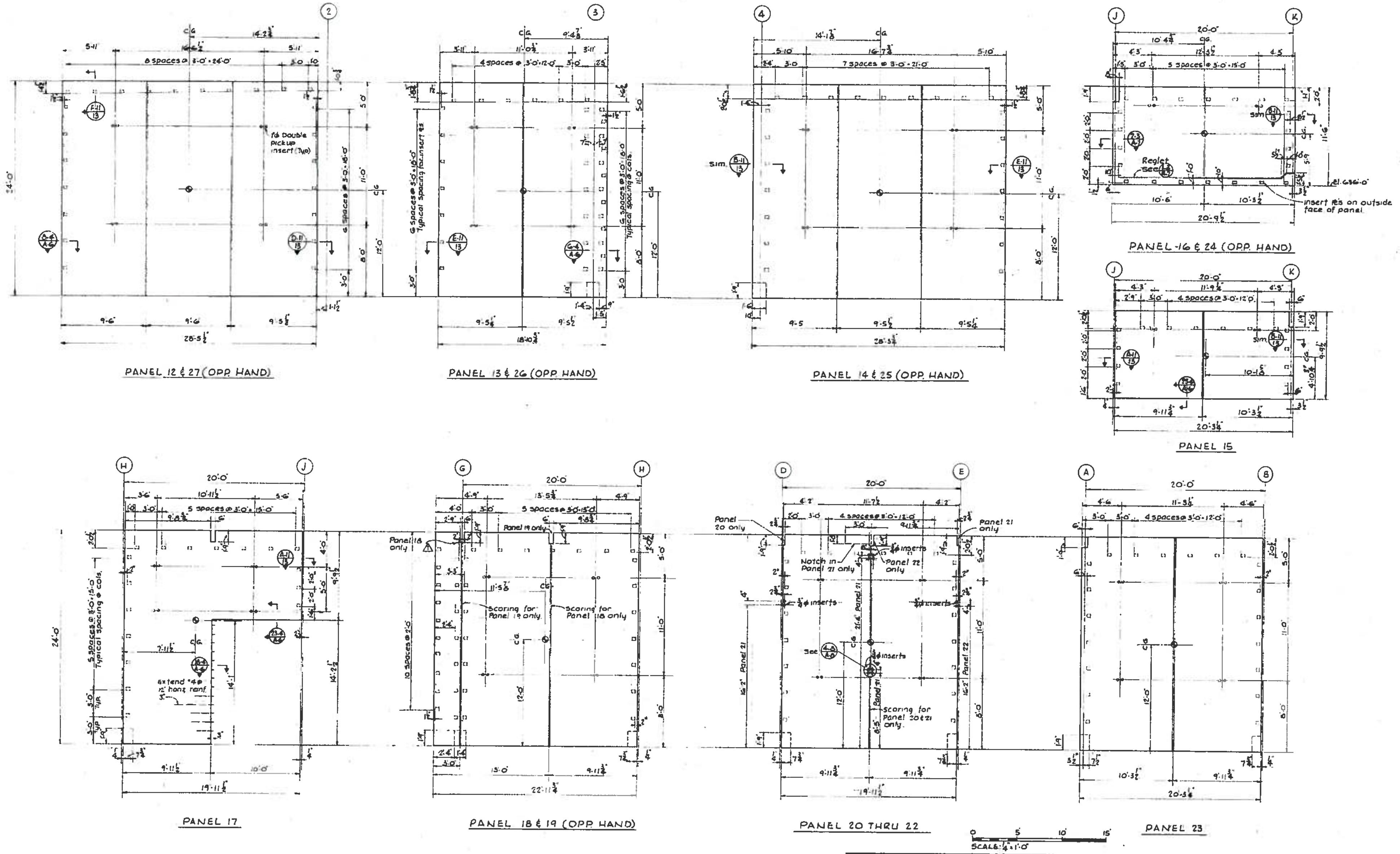
DATE: FEB. 19 1963
SCALE: 1/4"=1'-0"
DESIGNED BY: RW
DRAWN BY: TT
CHECKED BY: JBL

GARRETSON-ELMENDORF-KLEIN-REIBIN
ARCHITECTS · ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA
APPROVED BY: [signature]
JOB NO. 349

EXTERIOR PRECAST CONC. WALL DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA

DWG. NO. S11
REV. NO. 1

PLZ63-341-027JA



AS BUILT
BY TPL DATE 5-29-64

No.	DATE	REVISION	BY
△	5-29-64	As Built Corrections	RL

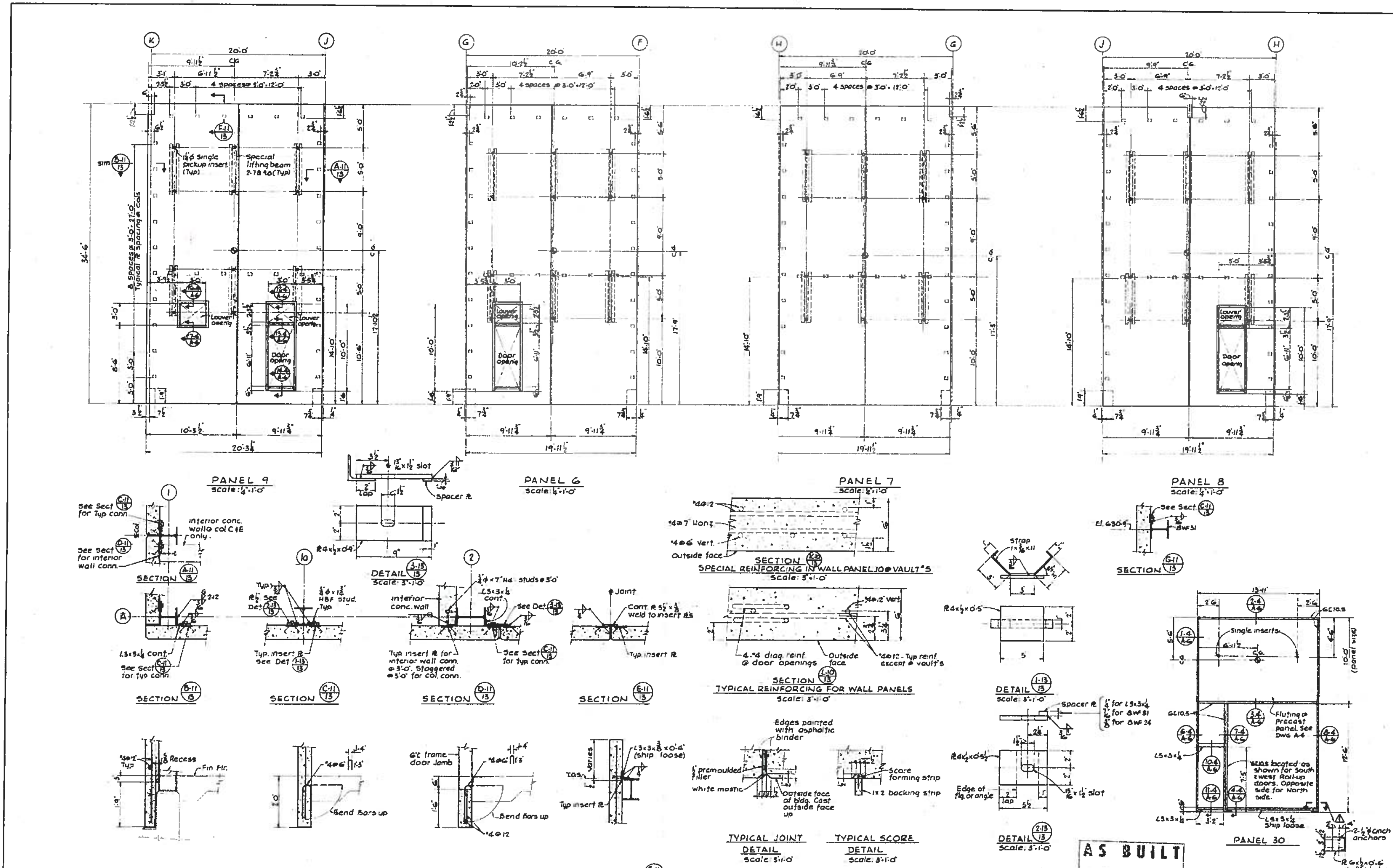
DATE: FEB. 19, 1963
SCALE: 1/4" = 1'-0"
DESIGNED BY: RW
DRAWN BY: TT
CHECKED BY: T.B.L.

GARRETSON-ELMENDORF-KLEIN-REIBIN
ARCHITECTS • ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA
APPROVED BY: [signature]
JOB NO. 349

EXTERIOR PRECAST CONC WALL DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE, CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA

DWG. NO. S12
REV. NO. △

PLZ63-341-028JA



No	DATE	REVISION	BY
△	5-29-64	As Built Corrections	RL

DATE: FEB. 19, 1963
SCALE: 1/4"=1'-0" & AS SHOWN
DESIGNED BY: RW
DRAWN BY: TT
CHECKED BY: T.B.L.
APPROVED BY: [signature]
JOB NO. 349

GARRETSON · ELMENDORF · KLEIN · REIBIN
ARCHITECTS · ENGINEERS
214 MARKET STREET, SAN FRANCISCO 11, CALIFORNIA

EXTERIOR PRECAST CONC. WALL DETAILS
PULSED ENERGY RESEARCH BUILDING NO. 184
LIVERMORE, CALIFORNIA
UNITED STATES ATOMIC ENERGY COMMISSION
2111 BANCROFT WAY, BERKELEY 4, CALIFORNIA

DWG. NO. S13
REV. NO. △

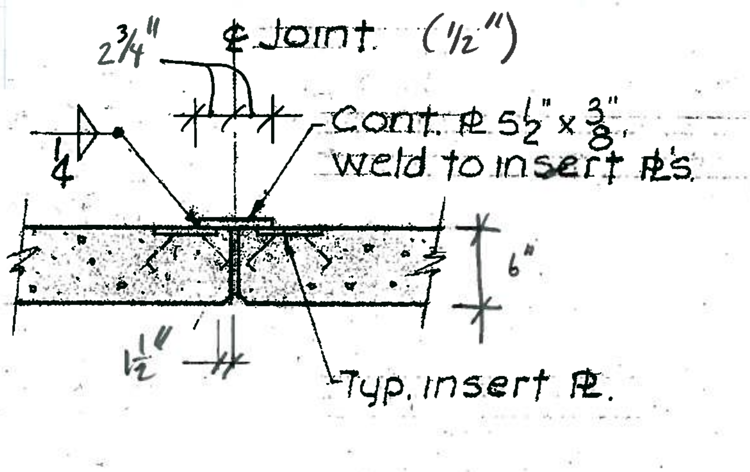
PLZ63-341-029JA

Degenkolb

Subject:	Job Number:	Date:	
Job:	By:	Section:	
	Checked By:	Page	of

PRECAST PANEL INSERT CAPACITY

PANEL - PANEL CONN

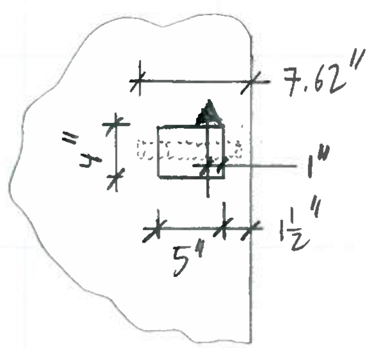


$1'' \times 3/16''$ STRAP

$A = 0.1875\ in^2$

EQUIV ROUND BAR $= 0.49''\phi$

USE $1/2''\phi$ FOR CALC.



WELD

$$\frac{1.39\ k/in/16^{TH} \times 4^{16THS} \times 4''}{0.75} = 30^k$$

CONC. ANCHORAGE

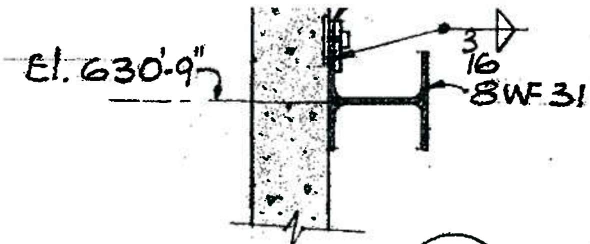
ACI APPENDIX D

≈ EQUIVALENT TO TWO ANCHORS - ONE @ 2.5" EDGE DIST & ONE @ 5.5" EDGE DIST

$V_{cbg} = 4057^{\#}$ (SEE SPRD SHT)

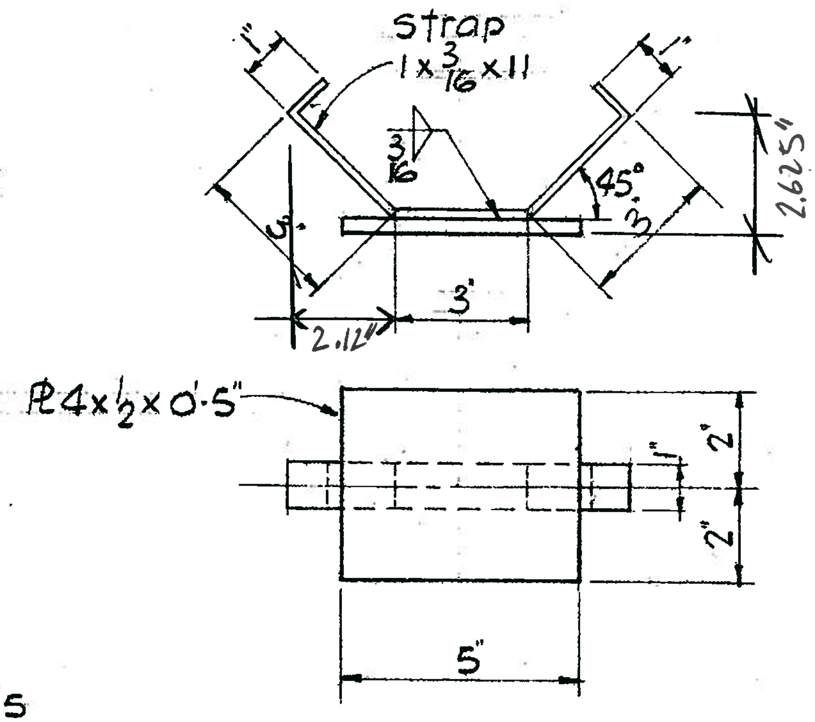
STRAP

$V_n = (2)(0.6)(44)(1'' \times 3/16'') = 9.9^k$



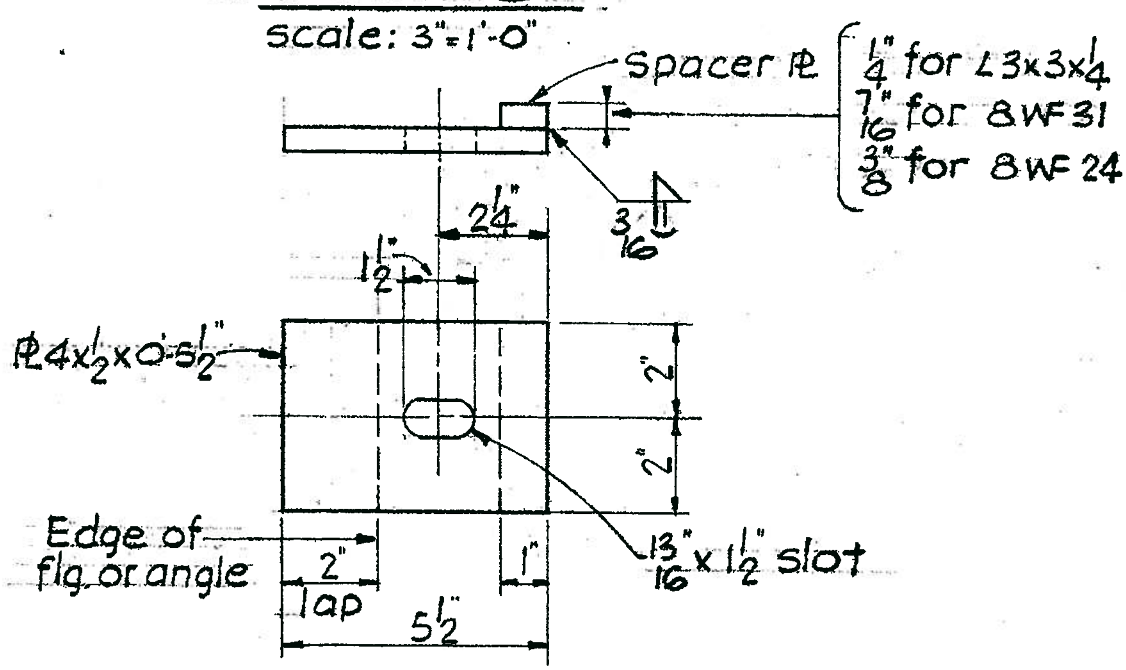
SECTION G-11/13

5



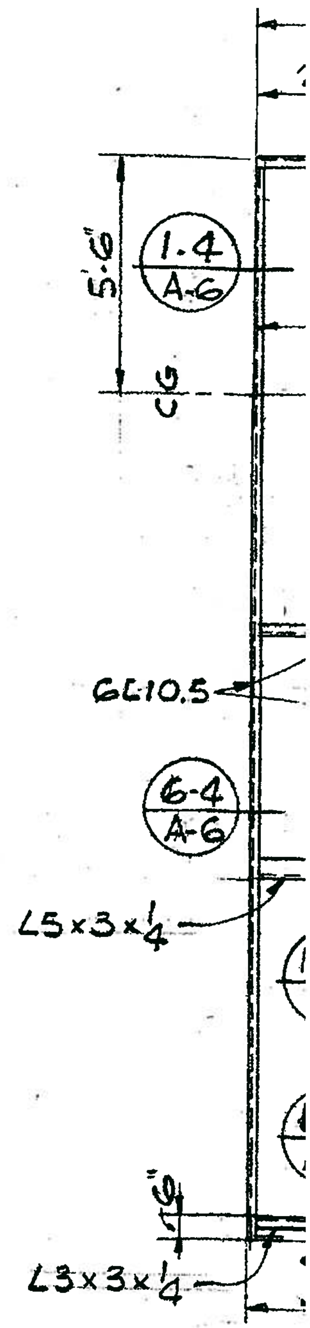
einf.
ault's

DETAIL 1-13/13
scale: 3"=1'-0"



strip

strip



DETAIL 2-13/13
scale: 3"=1'-0"

AS BUILT

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Panel-Column Precast Connection	**Job Number:** B31891012.00	**Date:** 12.20.13
Job: LLNL B341	**By:**	**Section:**
	Checked By:	**Page/of:**

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel-Panel Connection**
Loading:	**Shear along panel edge**

Cast-In-Place Anchor Properties:

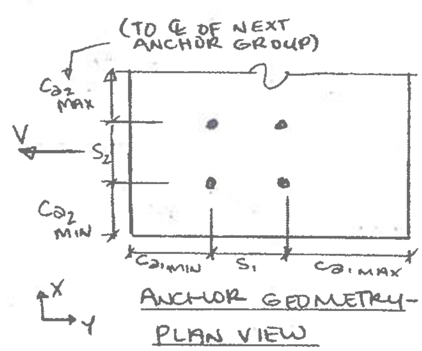
Material:	F1554 Gr. 36	
Diameter:	3/8" Φ	
Nut Type:	A563 Hex	
Supplementary Reinforcement (between anchor and edge):	None or <#4	
Anchor Yield Stress, f_y:	44	ksi
Anchor Tensile Stress, f_u:	62	ksi
d_o =	0.188	in
Effective Embedment Depth of Anchor, h_{ef}:	2.10	in
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef}	N/A	in *(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support?	No	
Min Thickness of Steel Attachment :	N/A	in. *(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable:	0.00	in.^{2} *(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable:	0.00	in. *(Input '0.00' if not used)*
Embedded nut width, if applicable:	0.00	in

Concrete Data:

f'_c =	4000	psi
Concrete Type =	Normal Weight	
λ =	1.00	*ACI 8.6.1*
Concrete Performance:	Cracking at Service Loads	
Concrete Depth, h_a:	6	in

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1:	3	in
Anchor Spacing in X-Direction, s_2:	0	in
Number of Anchor Rows in Y-Direction, n_y:	2	
Number of Anchor Rows in X-Direciton, n_x:	1	
Total Number of anchors in group, n:	2	
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}:	100.00	in
Y-Dir Edge Min. Edge Dist., c_{a1_min}:	2.5	in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}:	36.0	in
X-Dir Min. Edge Dist., c_{a2_min}:	36.0	in
Critical edge distance from ACI D.8.6, c_{ac}:	8.4	in
Dist from centroid of bolt group to applied tension load, e'_N*=	0.0	in
Diist from centroid of bolt group to applied shear load, e'_V* =	1.5	in
Can Shear Breakout Occur in Y-direction?	Y	
Can Shear Breakout Occur in X-direction?	N	



**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point =	0	lbs
Max Shear at Conn Point =	1000	lbs
Direction of Shear Loading =	X-Dir	
Required Incr. in Tension Demand =**	1.0	
Required Incr. in Shear Demand =**	1.0	

Tension Demand:

N_u = Tension at Anchor Group = 0 lbs

Shear Demand:

V_u = Shear at Anchor Group= 1000 lbs

****Required Demand Increases:**

Note: For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Panel-Column Precast Connection	**Job Number:** B31891012.00	**Date:**	12.20.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel-Panel Connection**
Loading:	**Shear along panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uta}$ — *ACI Eqn (D-3)*

Φ =	0.75	
A_{se} =	0.078	in^2
f_{ut} =	62000	psi
ΦN_{sa} =	**7254**	**lbs**
N_u =	0	lbs
$N_u / \Phi N_{sa}$ =	$\boxed{0.00}$	

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-4)*

$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-5)*

$\Psi_{ec,N}$	1.00	*Eqn (D-9)*
$\Psi_{ed,N}$	0.94	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	1.00	*Section D.5.2.6*
$\Psi_{cp,N}$	1.00	*Does not apply to cast-in-place anchors*

Φ = 0.7

$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ OR $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$ — *ACI Eqn (D-7) or (D-8)*

k_c =	24	
N_b =	4619 lbs	
A_{Nco} =	$9 * h_{ef}^2$	*ACI Eqn (D-6)*
A_{Nco} =	40 in^2	*(Projected area for a single anchor without edge distance considered)*
A_{Nc} =	54 in^2	*(Proj. area for group of anchors with edge dist. considered; Not greater than $n * A_{Nco}$)*
		(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or N_{cbg} =	5950 lbs	*Strength of group of anchors*
0.75ΦNcb or 0.75ΦNcbg =	**3124 lbs**	
N_u =	**0 lbs**	
$N_u / \Phi N_{cbg}$ =	$\boxed{0.00}$	

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$		*ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$		*ACI Eqn (D-15)*
A_{brg} =	0.16 in^2	*From PCA Notes on ACI 318-05 Table 34-2*
Φ =	0.70	
N_p =	5248 lbs	
$\Psi_{c,P}$ =	1.00 in	
0.75ΦNpn =	**2755 lbs**	*Strength of a single anchor in group*
N_u =	**0 lbs**	*Demand of a single anchor in group*
$N_u / \Phi N_p$ =	$\boxed{0.00}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Panel-Column Precast Connection	**Job Number:** B31891012.00	**Date:** 12.20.13	
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel-Panel Connection**
Loading:	**Shear along panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

$\Phi N_{sb} = \Phi*\lambda*160*c_{a1}\sqrt{A_{brg}}\sqrt{f'c}$		*ACI Eqn (D-17)*
$\Phi N_{sbg} = \Phi*(1+ s/(6*c_{a1}))*N_{sb}$		*ACI Eqn (D-18)*
$\Phi =$	0.70	
Deep embed. close to edge in Y-dir?	No	*"Yes" if $c_{a1} < 0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No	*"Yes" if $c_{a2} < 0.4h_{ef}$*
For anchor group , is $s_2 < 6*c_{a1}$?	Yes	*For blowout of group in Y-direction*
For anchor group , is $s_1 < 6*c_{a2}$?	Yes	*For blowout of group in X-direction*
Does failure mode apply in either dir?	No	*Failure mode applies if $c_{a\_min} < 0.4h_{ef}$ and if $s < 6*c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	1.0	*Factor applies for single anchor if $c_{a2} < 3c_{a1}$ where $1.0 <= c_{a2}/c_{a1} <= 3.0$ and $c_{a1} = c_{a\_min}$*
$N_{sb} =$	N/A lbs	
$N_{sbg} =$	N/A lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A lbs**	
$N_u =$	**lbs**	
$N_u / \Phi N_{sb} =$	$\boxed{\text{N/A}}$	

SHEAR CAPACITY: **Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}**

Steel Strength of Anchor in Shear: ACI D.6.1 *(for headed bolts)*

$\Phi V_{sa} = \Phi*n*0.6*A_{se}*f_{ut}$			*ACI Eqn (D-20)*
$\Phi =$	1		
$A_{se} =$	0.078	in^2	
$f_{ut} =$	62000	psi	
$\Phi V_{sa} =$	5803	lbs	
$V_u =$	1000	lbs	
$V_u / \Phi V_{sa} =$	$\boxed{0.17}$		

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel-Panel Connection**
Loading:	**Shear along panel edge**

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} = \Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$		*ACI Eqn (D-21)*
$\Phi V_{cbg} = \Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ec,V}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$		*ACI Eqn (D-22)*
$V_b = \lambda^* 7^*(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$		*ACI Eqn (D-24)*
A_{Vc} = (Based on geometry)	$\leq n^* A_{Vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} = 4.5^* c_{a1}^2$		*Projected failure area in shear for a single anchor w/out considering edge dist*
Φ = 1.00		
ℓ_e = 1.5 in		*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V}$ = 1.0		*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):			Assume load is distributed equally between anchors (check front anchors):		
c_{a1} =	5.50 in		c_{a1} =	2.50 in	*(This case should not be considered if anchors in group are rigidly connected to support)*
V_b =	3748 lbs		V_b =	1149 lbs	
A_{Vc} =	99 in^2		A_{Vc} =	28 in^2	
A_{Vco} =	136 in^2		A_{Vco} =	28 in^2	
$\Psi_{ed,V}$ =	1.00	*Eqn (D-27) or (D-28)*	$\Psi_{ed,V}$ =	1.00	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.85	*Eqn (D-26)*	$\Psi_{ec,V}$ =	0.71	*Eqn (D-26)*
$\Psi_{h,V}$ =	1.17	*Eqn (D-29)*	$\Psi_{h,V}$ =	1.00	*Eqn (D-29)*
V_{cbg} =	5409 lbs		V_{cbg} =	1641 lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**4057 lbs**		**0.75Φ(Vcb_y or Vcbg_y)=**	**1231 lbs**	
V_u =	1000 lbs		V_u =	500 lbs	
$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.25}$		$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.41}$	

Shear Breakout in X-Direction:

Assume all load acts on back anchor (check group):			Assume load is distributed equally between anchors (check front anchors):		
c_{a2} =	36.00 in		c_{a2} =	36.00 in	*(This case should not be considered if anchors in group are rigidly connected to support)*
V_b =	62763 lbs		V_b =	62763 lbs	
A_{Vc} =	357 in^2		A_{Vc} =	357 in^2	
A_{Vco} =	5832 in^2		A_{Vco} =	5832 in^2	
$\Psi_{ed,V}$ =	0.71	*Eqn (D-27) or (D-28)*	$\Psi_{ed,V}$ =	0.90	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.97	*Eqn (D-26)*	$\Psi_{ec,V}$ =	0.97	*Eqn (D-26)*
$\Psi_{h,V}$ =	3.00	*Eqn (D-29)*	$\Psi_{h,V}$ =	3.00	*Eqn (D-29)*
V_{cbg} =	8006 lbs		V_{cbg} =	10093 lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**6004 lbs**		**0.75Φ(Vcb_x or Vcbg_x)=**	**7570 lbs**	
V_u =	1000 lbs		V_u =	1000 lbs	
$V_u / \Phi V_{cbg\_x}$ =	$\boxed{0.17}$		$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.13}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Panel-Column Precast Connection	**Job Number:** B31891012.00	**Date:**	12.20.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel-Panel Connection**
Loading:	**Shear along panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi * k_{cp} * N_{cb}$ *ACI Eqn (D-29); for single anchor*

$\Phi V_{cpg} = \Phi * k_{cp} * N_{cbg}$ *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} = 5950 lbs

Φ = 1.00

k_{cp} = 1.00

0.75ΦVcp or 0.75ΦVcpg = 4462 lbs

V_u = 1000 lbs

$V_u / \Phi V_{cp}$ = $\boxed{0.22}$

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:	Ductile Failure D/C		Non-Ductile Failure D/C*	
Steel Strength of Anchor in Tension:	0.00		0.00	
Concrete Breakout Strength of Anchor in Tension:	0.00		0.00	
Concrete Pullout Strength of Anchor in Tension:	0.00		0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:	N/A		N/A	
max	**0.00**		**0.00**	**< 1.0, OK**

Summary of Shear D/C Ratios:	Ductile Failure D/C		Non-Ductile Failure D/C*	
Steel Strength of Anchor in Shear:	**0.17**		0.43	
Concrete Breakout Strength of Anchor in Shear:	**0.41**	Non-Ductile Failure Governs	1.02	
Concrete Pryout Strength of Anchor in Shear:	**0.22**		0.56	
max	**0.41**		**1.02**	**> 1.0, Redes**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multiplied by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = $\boxed{\text{N/A}}$

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = $\boxed{\text{N/A}}$

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 0.8 in. *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **Spacing OK**

Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

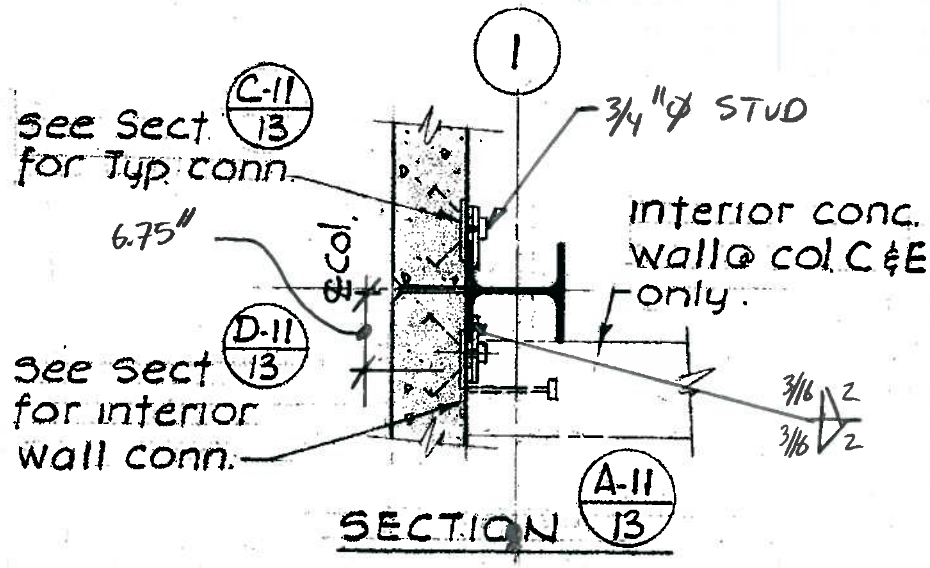
Check edge dist in direction of shear force: **Edge Dist OK**

Check edge dist perpendicular to shear force: **Edge Dist OK**

Degenkolb

Subject:		Job Number:	Date:
Job:		By:	Section:
		Checked By:	Page of

PANEL TO WF COL CONN



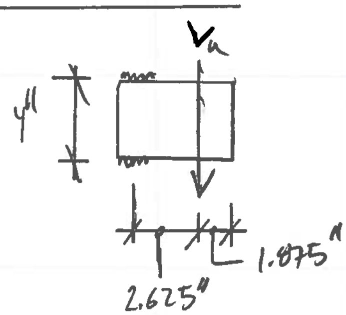
3/4"Ø STUD

ASSUME MILD STEEL $F_{nv} = 24^{ksi}$ (A307)

$$V_n = \frac{0.75^2 \pi}{4} \times 24^{ksi}$$

$$= 10.6^k$$

WELD TO COL



$A_{WELD} = (2)(2) = 4\ in^2/in$

$S_{WELD} = (2)(2)^2 = 8\ in^3/in$

$$\sigma_V = \frac{V}{4}$$

$$\sigma_b = \frac{V \times 2.625}{8}$$

$$\sigma_R = \sqrt{\frac{V^2}{16} + V^2 \times 0.108} = \frac{3 \times 1.39}{0.75} =$$

$$= 5.6\ ^k/_{in}$$

$$V = 13.6^k$$

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

CONC. ANCHOR

$V_{CBG} = 5125^{\#}$ (SEE SPREAD SHT)

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF column**
Loading:	**Shear along panel edge**

Cast-In-Place Anchor Properties:

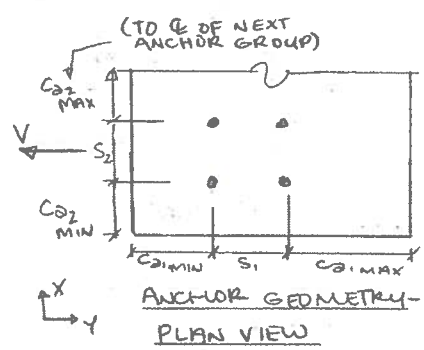
Material:	F1554 Gr. 36	
Diameter:	3/8" Φ	
Nut Type:	A563 Hex	
Supplementary Reinforcement (between anchor and edge):	None or <#4	
Anchor Yield Stress, f_y:	44	ksi
Anchor Tensile Stress, f_u:	62	ksi
d_o =	**0.188**	in
Effective Embedment Depth of Anchor, h_{ef}:	**2.10**	in
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef}	**N/A**	in *(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support?	**No**	
Min Thickness of Steel Attachment :	**N/A**	in. *(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable:	**0.00**	$in.^2$ *(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable:	**0.00**	in. *(Input '0.00' if not used)*
Embedded nut width, if applicable:	**0.00**	in

Concrete Data:

f'_c =	**4000**	psi
Concrete Type =	Normal Weight	
λ =	1.00	*ACI 8.6.1*
Concrete Performance:	Cracking at Service Loads	
Concrete Depth, h_a:	**6**	in

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1:	**3**	in
Anchor Spacing in X-Direction, s_2:	**0**	in
Number of Anchor Rows in Y-Direction, n_y:	**2**	
Number of Anchor Rows in X-Direciton, n_x:	**1**	
Total Number of anchors in group, n:	2	
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}:	**100.00**	in
Y-Dir Edge Min. Edge Dist., c_{a1_min}:	**3.8**	in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}:	**36.0**	in
X-Dir Min. Edge Dist., c_{a2_min}:	**36.0**	in
Critical edge distance from ACI D.8.6, c_{ac}:	8.4	in
Dist from centroid of bolt group to applied tension load, e'_N*=	**0.0**	in
Diist from centroid of bolt group to applied shear load, e'_V* =	**1.5**	in
Can Shear Breakout Occur in Y-direction?	**Y**	
Can Shear Breakout Occur in X-direction?	**N**	



**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point =	**0**	lbs
Max Shear at Conn Point =	**1000**	lbs
Direction of Shear Loading =	X-Dir	
Required Incr. in Tension Demand =**	**1.0**	
Required Incr. in Shear Demand =**	**1.0**	

Tension Demand:

N_u = Tension at Anchor Group = **0** lbs

Shear Demand:

V_u = Shear at Anchor Group= **1000** lbs

****Required Demand Increases:**

Note: For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF column**
Loading:	**Shear along panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uta}$ *ACI Eqn (D-3)*

$\Phi =$ 0.75
$A_{se} =$ 0.078 in^2
$f_{ut} =$ 62000 psi
$\Phi N_{sa} =$ 7254 lbs

$N_u =$ 0 lbs
$N_u / \Phi N_{sa} =$ $\boxed{0.00}$

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ *ACI Eq (D-4)*
$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ *ACI Eq (D-5)*
$\Phi =$ 0.7
$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ OR $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$
$k_c =$ 24 *ACI Eqn (D-7) or (D-8)*
$N_b =$ 4619 lbs
$A_{Nco} =$ $9 * h_{ef}^2$ *ACI Eqn (D-6)*
$A_{Nco} =$ 40 in^2 *(Projected area for a single anchor without edge distance considered)*
$A_{Nc} =$ 59 in^2 *(Proj. area for group of anchors with edge dist. considered; Not greater than $n * A_{Nco}$)*
(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or $N_{cbg} =$ 6819 lbs *Strength of group of anchors*
$0.75\Phi N_{cb}$ or $0.75\Phi N_{cbg} =$ 3580 lbs
$N_u =$ 0 lbs
$N_u / \Phi N_{cbg} =$ $\boxed{0.00}$

$\Psi_{ec,N}$	1.00	*Eqn (D-9)*
$\Psi_{ed,N}$	1.00	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	1.00	*Section D.5.2.6*
$\Psi_{cp,N}$	1.00	*Does not apply to cast-in-place anchors*

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$ *ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$ *ACI Eqn (D-15)*
$A_{brg} =$ 0.16 in^2 *From PCA Notes on ACI 318-05 Table 34-2*
$\Phi =$ 0.70
$N_p =$ 5248 lbs
$\Psi_{c,P} =$ 1.00 in
$0.75\Phi N_{pn} =$ 2755 lbs *Strength of a single anchor in group*
$N_u =$ 0 lbs *Demand of a single anchor in group*
$N_u / \Phi N_p =$ $\boxed{0.00}$

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Panel-Column Precast Connection	**Job Number:** B31891012.00	**Date:** 12.20.13
Job: LLNL B341	**By:**	**Section:**
	Checked By:	**Page/of:**

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF column**
Loading:	**Shear along panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

$\Phi N_{sb} = \Phi * \lambda * 160 * c_{a1} \sqrt{A_{brg}} \sqrt{f'c}$			*ACI Eqn (D-17)*
$\Phi N_{sbg} = \Phi * (1 + s/(6 * c_{a1})) * N_{sb}$			*ACI Eqn (D-18)*
$\Phi =$	0.70		
Deep embed. close to edge in Y-dir?	No		*"Yes" if $c_{a1} < 0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No		*"Yes" if $c_{a2} < 0.4h_{ef}$*
For anchor group , is $s_2 < 6 * c_{a1}$?	Yes		*For blowout of group in Y-direction*
For anchor group , is $s_1 < 6 * c_{a2}$?	Yes		*For blowout of group in X-direction*
Does failure mode apply in either dir?	No		*Failure mode applies if $c_{a\_min} < 0.4h_{ef}$ and if $s < 6 * c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	1.0		*Factor applies for single anchor if $c_{a2} < 3c_{a1}$ where $1.0 <= c_{a2}/c_{a1} <= 3.0$ and $c_{a1} = c_{a\_min}$*
$N_{sb} =$	N/A	lbs	
$N_{sbg} =$	N/A	lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A**	**lbs**	
$N_u =$		**lbs**	
$N_u / \Phi N_{sb} =$	**N/A**		

SHEAR CAPACITY: Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}

Steel Strength of Anchor in Shear: ACI D.6.1

(for headed bolts)

$\Phi V_{sa} = \Phi * n * 0.6 * A_{se} * f_{ut}$			*ACI Eqn (D-20)*
$\Phi =$	1		
$A_{se} =$	0.078	in^2	
$f_{ut} =$	62000	psi	
$\Phi V_{sa} =$	**5803**	**lbs**	
$V_u =$	**1000**	**lbs**	
$V_u / \Phi V_{sa} =$	**0.17**		

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF column**
Loading:	**Shear along panel edge**

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} =$	$\Phi * A_{Vc}/A_{Vco} * \Psi_{ed,V} * \Psi_{c,V} * V_b$	*ACI Eqn (D-21)*
$\Phi V_{cbg} =$	$\Phi * A_{Vc}/A_{Vco} * \Psi_{ec,V} * \Psi_{ed,V} * \Psi_{c,V} * V_b$	*ACI Eqn (D-22)*
$V_b =$	$\lambda * 7 * (\ell_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_{a1})^{1.5}$	*ACI Eqn (D-24)*
$A_{vc} =$	(Based on geometry) $\leq n * A_{vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} =$	$4.5 * c_{a1}^2$	*Projected failure area in shear for a single anchor w/out considering edge dist*
$\Phi =$	1.00	
$\ell_e =$	1.5 in	*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V} =$	1.0	*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):

$c_{a1} =$	6.75	in	
$V_b =$	5096	lbs	
$A_{Vc} =$	122	in^2	
$A_{Vco} =$	205	in^2	
$\Psi_{ed,V} =$	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.87		*Eqn (D-26)*
$\Psi_{h,V} =$	1.30		*Eqn (D-29)*
$V_{cbg} =$	6833	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**5125**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.20}$		

Assume load is distributed equally between anchors (check front anchors):

(This case should not be considered if anchors in group are rigidly connected to support)

$c_{a1} =$	3.75	in	
$V_b =$	2110	lbs	
$A_{Vc} =$	63	in^2	
$A_{Vco} =$	63	in^2	
$\Psi_{ed,V} =$	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.79		*Eqn (D-26)*
$\Psi_{h,V} =$	1.00		*Eqn (D-29)*
$V_{cbg} =$	3332	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**2499**	**lbs**	
$V_u =$	500	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.20}$		

Shear Breakout in X-Direction:

Assume all load acts on back anchor (check group):

$c_{a2} =$	36.00	in	
$V_b =$	62763	lbs	
$A_{Vc} =$	365	in^2	
$A_{Vco} =$	5832	in^2	
$\Psi_{ed,V} =$	0.72		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.97		*Eqn (D-26)*
$\Psi_{h,V} =$	3.00		*Eqn (D-29)*
$V_{cbg} =$	8253	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**6190**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_x} =$	$\boxed{0.16}$		

Assume load is distributed equally between anchors (check front anchors):

(This case should not be considered if anchors in group are rigidly connected to support)

$c_{a2} =$	36.00	in	
$V_b =$	62763	lbs	
$A_{Vc} =$	365	in^2	
$A_{Vco} =$	5832	in^2	
$\Psi_{ed,V} =$	0.90		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.97		*Eqn (D-26)*
$\Psi_{h,V} =$	3.00		*Eqn (D-29)*
$V_{cbg} =$	10305	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**7729**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.13}$		

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF column**
Loading:	**Shear along panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi^* k_{cp}^* N_{cb}$ *ACI Eqn (D-29); for single anchor*

$\Phi V_{cpg} = \Phi^* k_{cp}^* N_{cbg}$ *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} = 6819 lbs

Φ = 1.00

k_{cp} = 1.00

0.75ΦVcp or 0.75ΦVcpg = 5114 lbs

V_u = 1000 lbs

$V_u / \Phi V_{cp}$ = $\boxed{0.20}$

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:	**Ductile Failure D/C**		**Non-Ductile Failure D/C***	
Steel Strength of Anchor in Tension:	0.00		0.00	
Concrete Breakout Strength of Anchor in Tension:	0.00		0.00	
Concrete Pullout Strength of Anchor in Tension:	0.00		0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:	N/A		N/A	
max	**0.00**		**0.00**	**< 1.0, OK**

Summary of Shear D/C Ratios:				
Steel Strength of Anchor in Shear:	**0.17**		0.43	
Concrete Breakout Strength of Anchor in Shear:	**0.20**	Non-Ductile Failure Governs	0.50	
Concrete Pryout Strength of Anchor in Shear:	**0.20**		0.49	
max	**0.20**		**0.50**	**< 1.0, OK**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multiplied by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 0.8 in. *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **Spacing OK**

Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

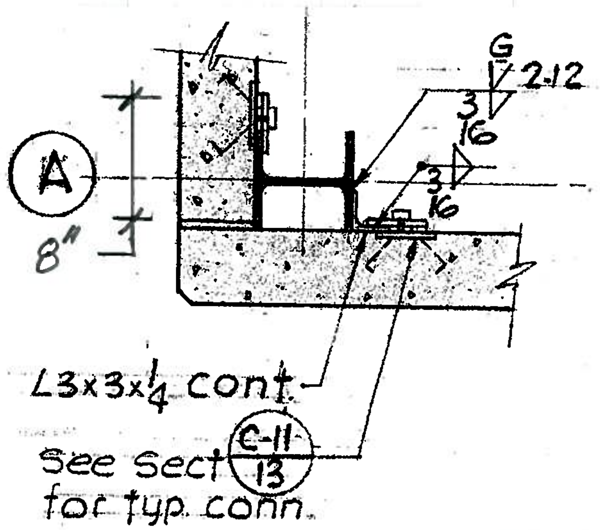
Check edge dist in direction of shear force: **Edge Dist OK**

Check edge dist perpendicular to shear force: **Edge Dist OK**

Degenkolb

Subject:		Job Number:	Date:
Job:		By:	Section:
		Checked By:	Page of

PANEL - COL @ CORNER



WELDS & STUDS SAME AS COL- PANEL

CONC. ANCHORAGE

$V_{CAP} = 8790\#$

CONTRAS

SEE SARD. EA71

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF Corner Column**
Loading:	**Shear along panel edge**

Cast-In-Place Anchor Properties:

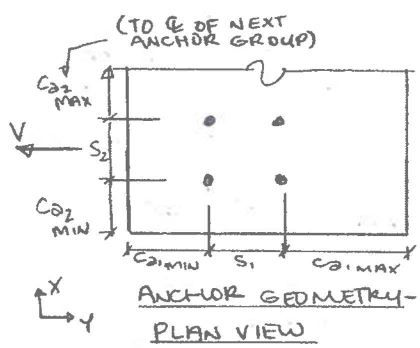
Material:	F1554 Gr. 36	
Diameter:	3/8" Φ	
Nut Type:	A563 Hex	
Supplementary Reinforcement (between anchor and edge):	None or <#4	
Anchor Yield Stress, f_y:	44	ksi
Anchor Tensile Stress, f_u:	62	ksi
d_o =	**0.188**	in
Effective Embedment Depth of Anchor, h_{ef}:	**2.10**	in
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef}	**N/A**	in *(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support?	**No**	
Min Thickness of Steel Attachment :	**N/A**	in. *(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable:	**0.00**	in.^{2} *(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable:	**0.00**	in. *(Input '0.00' if not used)*
Embedded nut width, if applicable:	**0.00**	in

Concrete Data:

f'_c =	**4000**	psi
Concrete Type =	Normal Weight	
λ =	1.00	*ACI 8.6.1*
Concrete Performance:	Cracking at Service Loads	
Concrete Depth, h_a:	**6**	in

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1:	**3**	in
Anchor Spacing in X-Direction, s_2:	**0**	in
Number of Anchor Rows in Y-Direction, n_y:	**2**	
Number of Anchor Rows in X-Direciton, n_x:	**1**	
Total Number of anchors in group, n:	**2**	
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}:	**100.00**	in
Y-Dir Edge Min. Edge Dist., c_{a1_min}:	**8.0**	in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}:	**36.0**	in
X-Dir Min. Edge Dist., c_{a2_min}:	**36.0**	in
Critical edge distance from ACI D.8.6, c_{ac}:	8.4	in
Dist from centroid of bolt group to applied tension load, e'_N*=	**0.0**	in
Diist from centroid of bolt group to applied shear load, e'_V* =	**1.5**	in
Can Shear Breakout Occur in Y-direction?	**Y**	
Can Shear Breakout Occur in X-direction?	**N**	



**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point =	0	lbs
Max Shear at Conn Point =	1000	lbs
Direction of Shear Loading =	X-Dir	
Required Incr. in Tension Demand =**	1.0	
Required Incr. in Shear Demand =**	1.0	

Tension Demand:

N_u = Tension at Anchor Group = 0 lbs

Shear Demand:

V_u = Shear at Anchor Group= 1000 lbs

****Required Demand Increases:**

Note: *For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.*
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF Corner Column**
Loading:	**Shear along panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: **Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}**

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uta}$ — *ACI Eqn (D-3)*

$\Phi =$ 0.75
$A_{se} =$ 0.078 in^2
$f_{ut} =$ 62000 psi
$\Phi N_{sa} =$ **7254 lbs**
$N_u =$ 0 lbs
$N_u / \Phi N_{sa} =$ $\boxed{0.00}$

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-4)*
$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-5)*

$\Psi_{ec,N}$	1.00	*Eqn (D-9)*
$\Psi_{ed,N}$	1.00	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	1.00	*Section D.5.2.6*
$\Psi_{cp,N}$	1.00	*Does not apply to cast-in-place anchors*

$\Phi =$ 0.7
$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ OR $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$
$k_c =$ 24 — *ACI Eqn (D-7) or (D-8)*
$N_b =$ 4619 lbs
$A_{Nco} =$ $9 * h_{ef}^2$ — *ACI Eqn (D-6)*
$A_{Nco} =$ 40 in^2 — *(Projected area for a single anchor without edge distance considered)*
$A_{Nc} =$ 59 in^2 — *(Proj. area for group of anchors with edge dist. considered; Not greater than $n*A_{Nco}$)*
(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or $N_{cbg} =$ 6819 lbs — *Strength of group of anchors*
$0.75\Phi N_{cb}$ or $0.75\Phi N_{cbg}$ = 3580 lbs
$N_u =$ **0 lbs**
$N_u / \Phi N_{cbg} =$ $\boxed{0.00}$

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$ — *ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$ — *ACI Eqn (D-15)*
$A_{brg} =$ 0.16 in^2 — *From PCA Notes on ACI 318-05 Table 34-2*
$\Phi =$ 0.70
$N_p =$ 5248 lbs
$\Psi_{c,P} =$ 1.00 in
$0.75\Phi N_{pn}$ = 2755 lbs — *Strength of a single anchor in group*
$N_u =$ **0 lbs** — *Demand of a single anchor in group*
$N_u / \Phi N_p =$ $\boxed{0.00}$

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF Corner Column**
Loading:	**Shear along panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

$\Phi N_{sb} = \Phi * \lambda * 160 * c_{a1} \sqrt{A_{brg}} \sqrt{f'c}$		*ACI Eqn (D-17)*
$\Phi N_{sbg} = \Phi * (1 + s/(6 * c_{a1})) * N_{sb}$		*ACI Eqn (D-18)*
Φ =	0.70	
Deep embed. close to edge in Y-dir?	No	*"Yes" if $c_{a1} < 0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No	*"Yes" if $c_{a2} < 0.4h_{ef}$*
For anchor group , is $s_2 < 6 * c_{a1}$?	Yes	*For blowout of group in Y-direction*
For anchor group , is $s_1 < 6 * c_{a2}$?	Yes	*For blowout of group in X-direction*
Does failure mode apply in either dir?	No	*Failure mode applies if $c_{a\_min} < 0.4h_{ef}$ and if $s < 6 * c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	1.0	*Factor applies for single anchor if $c_{a2} < 3c_{a1}$ where $1.0 <= c_{a2}/c_{a1} <= 3.0$ and $c_{a1} = c_{a\_min}$*
N_{sb} =	N/A lbs	
N_{sbg} =	N/A lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A lbs**	
N_u =	lbs	
$N_u / \Phi N_{sb}$ =	N/A	

SHEAR CAPACITY: **Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}**

Steel Strength of Anchor in Shear: ACI D.6.1 *(for headed bolts)*

$\Phi V_{sa} = \Phi * n * 0.6 * A_{se} * f_{ut}$			*ACI Eqn (D-20)*
Φ =	1		
A_{se} =	0.078	in^2	
f_{ut} =	62000	psi	
ΦV_{sa} =	**5803**	**lbs**	
V_u =	**1000**	**lbs**	
$V_u / \Phi V_{sa}$ =	**0.17**		

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF Corner Column**
Loading:	**Shear along panel edge**

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} =$	$\Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$	*ACI Eqn (D-21)*
$\Phi V_{cbg} =$	$\Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ec,V}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$	*ACI Eqn (D-22)*
$V_b =$	$\lambda^*7^*(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$	*ACI Eqn (D-24)*
$A_{Vc} =$	(Based on geometry) $\leq n^*A_{Vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} =$	$4.5^*c_{a1}^2$	*Projected failure area in shear for a single anchor w/out considering edge dist*
$\Phi =$	1.00	
$\ell_e =$	1.5 in	*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V} =$	1.0	*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):

$c_{a1} =$	11.00 in	
$V_b =$	10601 lbs	
$A_{Vc} =$	198 in^2	
$A_{Vco} =$	545 in^2	
$\Psi_{ed,V} =$	1.00	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.92	*Eqn (D-26)*
$\Psi_{h,V} =$	1.66	*Eqn (D-29)*
$V_{cbg} =$	11720 lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**8790 lbs**	
$V_u =$	1000 lbs	
$V_u / \Phi V_{cbg\_y} =$	**0.11**	

Assume load is distributed equally between anchors (check front anchors): *(This case should not be considered if anchors in group are rigidly connected to support)*

$c_{a1} =$	8.00 in	
$V_b =$	6575 lbs	
$A_{Vc} =$	144 in^2	
$A_{Vco} =$	288 in^2	
$\Psi_{ed,V} =$	1.00	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.89	*Eqn (D-26)*
$\Psi_{h,V} =$	1.41	*Eqn (D-29)*
$V_{cbg} =$	8265 lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**6199 lbs**	
$V_u =$	500 lbs	
$V_u / \Phi V_{cbg\_y} =$	**0.08**	

Shear Breakout in X-Direction:

Assume all load acts on back anchor (check group):

$c_{a2} =$	36.00 in	
$V_b =$	62763 lbs	
$A_{Vc} =$	390 in^2	
$A_{Vco} =$	5832 in^2	
$\Psi_{ed,V} =$	0.74	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.97	*Eqn (D-26)*
$\Psi_{h,V} =$	3.00	*Eqn (D-29)*
$V_{cbg} =$	9120 lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**6840 lbs**	
$V_u =$	1000 lbs	
$V_u / \Phi V_{cbg\_x} =$	**0.15**	

Assume load is distributed equally between anchors (check front anchors): *(This case should not be considered if anchors in group are rigidly connected to support)*

$c_{a2} =$	36.00 in	
$V_b =$	62763 lbs	
$A_{Vc} =$	390 in^2	
$A_{Vco} =$	5832 in^2	
$\Psi_{ed,V} =$	0.90	*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	0.97	*Eqn (D-26)*
$\Psi_{h,V} =$	3.00	*Eqn (D-29)*
$V_{cbg} =$	11026 lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**8269 lbs**	
$V_u =$	1000 lbs	
$V_u / \Phi V_{cbg\_y} =$	**0.12**	

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Panel-Column Precast Connection	**Job Number:**	B31891012.00	**Date:**	12.20.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Panel to WF Corner Column**
Loading:	**Shear along panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi * k_{cp} * N_{cb}$ — *ACI Eqn (D-29); for single anchor*

$\Phi V_{cpg} = \Phi * k_{cp} * N_{cbg}$ — *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} = 6819 lbs

Φ = 1.00

k_{cp} = 1.00

0.75ΦVcp or 0.75ΦVcpg = 5114 lbs

V_u = 1000 lbs

$V_u / \Phi V_{cp}$ = $\boxed{0.20}$

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:

	Ductile Failure D/C	Non-Ductile Failure D/C*	
Steel Strength of Anchor in Tension:	0.00	0.00	
Concrete Breakout Strength of Anchor in Tension:	0.00	0.00	
Concrete Pullout Strength of Anchor in Tension:	0.00	0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:	N/A	N/A	
max	**0.00**	**0.00**	**< 1.0, OK**

Summary of Shear D/C Ratios:

	Ductile Failure D/C		Non-Ductile Failure D/C*	
Steel Strength of Anchor in Shear:	**0.17**		0.43	
Concrete Breakout Strength of Anchor in Shear:	**0.15**		0.37	
Concrete Pryout Strength of Anchor in Shear:	**0.20**	Non-Ductile Failure Governs	0.49	
max	**0.20**		**0.49**	**< 1.0, OK**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multipled by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 0.8 in. — *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **Spacing OK**

Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. — *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

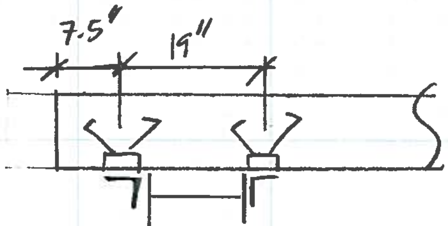
Check edge dist in direction of shear force: **Edge Dist OK**

Check edge dist perpendicular to shear force: **Edge Dist OK**

Degenkolb

Subject:		Job Number:	Date:
Job:		By:	Section:
		Checked By:	Page of

DOUBLE SIDED CONN TO COL - LINE 3



STUDS = 2×10.6^k = 21.2^k

WELDS = 2×13.6^k = 27.2^k

CONC ANCHORAGE = $10000^{\#}$ SPRD SHT

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Precast Panel Connections	**Job Number:**	B31891012.00	**Date:**	12.4.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Double Sided Panel to Column Connection**
Loading:	**Shear along panel edge**

Cast-In-Place Anchor Properties:

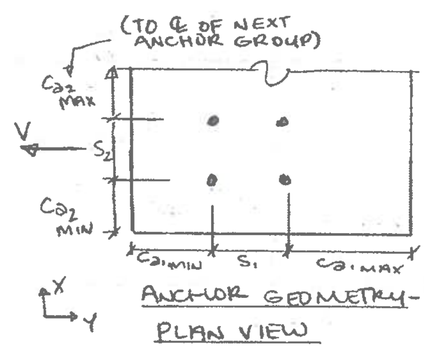
Material:	F1554 Gr. 36	
Diameter:	3/8" Φ	
Nut Type:	A563 Hex	
Supplementary Reinforcement (between anchor and edge):	None or <#4	
Anchor Yield Stress, f_y:	44	ksi
Anchor Tensile Stress, f_u:	62	ksi
d_o =	0.188	in
Effective Embedment Depth of Anchor, h_{ef}:	2.63	in
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef}	N/A	in *(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support?	No	
Min Thickness of Steel Attachment :	N/A	in. *(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable:	0.00	in.^{2} *(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable:	0.00	in. *(Input '0.00' if not used)*
Embedded nut width, if applicable:	0.00	in

Concrete Data:

f'_c =	4000	psi
Concrete Type =	Normal Weight	
λ =	1.00	*ACI 8.6.1*
Concrete Performance:	Cracking at Service Loads	
Concrete Depth, h_a:	6	in

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1:	3	in
Anchor Spacing in X-Direction, s_2:	0	in
Number of Anchor Rows in Y-Direction, n_y:	2	
Number of Anchor Rows in X-Direciton, n_x:	1	
Total Number of anchors in group, n:	2	
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}:	100.00	in
Y-Dir Edge Min. Edge Dist., c_{a1_min}:	26.5	in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}:	36.0	in
X-Dir Min. Edge Dist., c_{a2_min}:	36.0	in
Critical edge distance from ACI D.8.6, c_{ac}:	10.5	in
Dist from centroid of bolt group to applied tension load, e'_N*=	0.0	in
Diist from centroid of bolt group to applied shear load, e'_V* =	1.5	in
Can Shear Breakout Occur in Y-direction?	Y	
Can Shear Breakout Occur in X-direction?	N	



**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point =	0	lbs
Max Shear at Conn Point =	1000	lbs
Direction of Shear Loading =	X-Dir	
Required Incr. in Tension Demand =**	1.0	
Required Incr. in Shear Demand =**	1.0	

Tension Demand:

N_u = Tension at Anchor Group = 0 lbs

Shear Demand:

V_u = Shear at Anchor Group= 1000 lbs

****Required Demand Increases:**

Note: For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:**	12.4.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Double Sided Panel to Column Connection**
Loading:	**Shear along panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uta}$ — *ACI Eqn (D-3)*

Φ =	0.75	
A_{se} =	0.078	in^2
f_{ut} =	62000	psi
ΦN_{sa} =	**7254**	**lbs**
N_u =	0	lbs
$N_u / \Phi N_{sa}$ =	$\boxed{0.00}$	

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-4)*

$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-5)*

$\Psi_{ec,N}$	**1.00**	*Eqn (D-9)*
$\Psi_{ed,N}$	**1.00**	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	**1.00**	*Section D.5.2.6*
$\Psi_{cp,N}$	**1.00**	*Does not apply to cast-in-place anchors*

Φ = 0.7

$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ OR $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$

k_c =	24	*ACI Eqn (D-7) or (D-8)*
N_b =	6456 lbs	
A_{Nco} =	$9 * h_{ef}^2$	*ACI Eqn (D-6)*
A_{Nco} =	62 in^2	*(Projected area for a single anchor without edge distance considered)*
A_{Nc} =	86 in^2	*(Proj. area for group of anchors with edge dist. considered; Not greater than $n * A_{Nco}$)*
		(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or N_{cbg} =	8915 lbs	*Strength of group of anchors*
$0.75\Phi N_{cb}$ or $0.75\Phi N_{cbg}$ =	**4680 lbs**	
N_u =	**0 lbs**	
$N_u / \Phi N_{cbg}$ =	$\boxed{0.00}$	

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$		*ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$		*ACI Eqn (D-15)*
A_{brg} =	0.16 in^2	*From PCA Notes on ACI 318-05 Table 34-2*
Φ =	0.70	
N_p =	5248 lbs	
$\Psi_{c,P}$ =	1.00 in	
$0.75\Phi N_{pn}$ =	**2755 lbs**	*Strength of a single anchor in group*
N_u =	**0 lbs**	*Demand of a single anchor in group*
$N_u / \Phi N_p$ =	$\boxed{0.00}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:** 12.4.13
Job: LLNL B341	**By:**	**Section:**
	Checked By:	**Page/of:**

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Double Sided Panel to Column Connection**
Loading:	**Shear along panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

ΦN_{sb} =	$\Phi*\lambda*160*c_{a1}\sqrt{A_{brg}}\sqrt{f'c}$		*ACI Eqn (D-17)*
ΦN_{sbg} =	$\Phi*(1+ s/(6*c_{a1}))*N_{sb}$		*ACI Eqn (D-18)*
Φ =	0.70		
Deep embed. close to edge in Y-dir?	No		*"Yes" if $c_{a1}<0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No		*"Yes" if $c_{a2}<0.4h_{ef}$*
For anchor group , is $s_2<6*c_{a1}$?	Yes		*For blowout of group in Y-direction*
For anchor group , is $s_1<6*c_{a2}$?	Yes		*For blowout of group in X-direction*
Does failure mode apply in either dir?	No		*Failure mode applies if $c_{a\_min}<0.4h_{ef}$ and if $s<6*c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	0.6		*Factor applies for single anchor if $c_{a2}<3c_{a1}$ where $1.0<=c_{a2}/c_{a1}<=3.0$ and $c_{a1}=c_{a\_min}$*
N_{sb} =	N/A	lbs	
N_{sbg} =	N/A	lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A**	**lbs**	
N_u =		**lbs**	
$N_u / \Phi N_{sb}$ =	**N/A**		

SHEAR CAPACITY: Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}

Steel Strength of Anchor in Shear: ACI D.6.1 *(for headed bolts)*

ΦV_{sa} =	$\Phi*n*0.6*A_{se}*f_{ut}$		*ACI Eqn (D-20)*
Φ =	1		
A_{se} =	0.078	in^2	
f_{ut} =	62000	psi	
ΦV_{sa} =	**5803**	**lbs**	
V_u =	**1000**	**lbs**	
$V_u / \Phi V_{sa}$ =	**0.17**		

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:**	12.4.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Double Sided Panel to Column Connection**
Loading:	**Shear along panel edge**

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} = \Phi * A_{Vc}/A_{Vco} * \Psi_{ed,V} * \Psi_{c,V} * V_b$		*ACI Eqn (D-21)*
$\Phi V_{cbg} = \Phi * A_{Vc}/A_{Vco} * \Psi_{ec,V} * \Psi_{ed,V} * \Psi_{c,V} * V_b$		*ACI Eqn (D-22)*
$V_b = \lambda * 7 * (l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_{a1})^{1.5}$		*ACI Eqn (D-24)*
A_{Vc} = (Based on geometry)	$\le n * A_{Vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} = 4.5 * c_{a1}^2$		*Projected failure area in shear for a single anchor w/out considering edge dist*
Φ = 1.00		
l_e = 1.5 in		*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V}$ = 1.0		*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):

c_{a1} =	24.00	in	
V_b =	34164	lbs	
A_{Vc} =	432	in^2	
A_{Vco} =	2592	in^2	
$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.96		*Eqn (D-26)*
$\Psi_{h,V}$ =	2.45		*Eqn (D-29)*
V_{cbg} =	26779	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**20084**	**lbs**	
V_u =	1000	lbs	
$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.05}$		

Assume load is distributed equally between anchors (check front anchors):

(This case should not be considered if anchors in group are rigidly connected to support)

c_{a1} =	26.50	in	
V_b =	39638	lbs	
A_{Vc} =	432	in^2	
A_{Vco} =	3160	in^2	
$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.96		*Eqn (D-26)*
$\Psi_{h,V}$ =	2.57		*Eqn (D-29)*
V_{cbg} =	26880	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**20160**	**lbs**	
V_u =	500	lbs	
$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.02}$		

Shear Breakout in X-Direction:

Assume all load acts on back anchor (check group):

c_{a2} =	36.00	in	
V_b =	62763	lbs	
A_{Vc} =	501	in^2	
A_{Vco} =	5832	in^2	
$\Psi_{ed,V}$ =	0.85		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.97		*Eqn (D-26)*
$\Psi_{h,V}$ =	3.00		*Eqn (D-29)*
V_{cbg} =	13333	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**10000**	**lbs**	
V_u =	1000	lbs	
$V_u / \Phi V_{cbg\_x}$ =	$\boxed{0.10}$		

Assume load is distributed equally between anchors (check front anchors):

(This case should not be considered if anchors in group are rigidly connected to support)

c_{a2} =	36.00	in	
V_b =	62763	lbs	
A_{Vc} =	501	in^2	
A_{Vco} =	5832	in^2	
$\Psi_{ed,V}$ =	0.90		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	0.97		*Eqn (D-26)*
$\Psi_{h,V}$ =	3.00		*Eqn (D-29)*
V_{cbg} =	14164	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**10623**	**lbs**	
V_u =	1000	lbs	
$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.09}$		

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Precast Panel Connections**
Condition:	**Double Sided Panel to Column Connection**
Loading:	**Shear along panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi^* k_{cp}^* N_{cb}$ — *ACI Eqn (D-29); for single anchor*

$\Phi V_{cpg} = \Phi^* k_{cp}^* N_{cbg}$ — *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} =	8915	lbs
Φ =	1.00	
k_{cp} =	2.00	
0.75ΦVcp or 0.75ΦVcpg =	**13372**	**lbs**
V_u =	1000	lbs
$V_u / \Phi V_{cp}$ =	$\boxed{0.07}$	

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:

	Ductile Failure D/C	Non-Ductile Failure D/C*	
Steel Strength of Anchor in Tension:	0.00	0.00	
Concrete Breakout Strength of Anchor in Tension:	0.00	0.00	
Concrete Pullout Strength of Anchor in Tension:	0.00	0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:	N/A	N/A	
max	**0.00**	**0.00**	**< 1.0, OK**

Summary of Shear D/C Ratios:

	Ductile Failure D/C		Non-Ductile Failure D/C*
Steel Strength of Anchor in Shear:	**0.17**	Ductile Failure Governs	0.43
Concrete Breakout Strength of Anchor in Shear:	**0.10**		0.25
Concrete Pryout Strength of Anchor in Shear:	**0.07**		0.19
max	**0.17**	**< 1.0, OK**	**0.43**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multipled by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 0.8 in. — *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **Spacing OK**

Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. — *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

Check edge dist in direction of shear force: **Edge Dist OK**

Check edge dist perpendicular to shear force: **Edge Dist OK**

Evaluation of Braces on Line 2a

December 2013

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

EVALUATE BRACES ON LINE 2a

$P_u = 10.4^K$

2L 3x2x3/16

$K\ell = (1)(9.6^2 + 20^2)^{1/2} = 22'$

m-factor = 5

$P_{cr} = 3.6^K$ NEXT PAGE

$DCR_{MAX} = 10.4^K / (5 \times 3.6^K) = 0.58$ OK BUCKLING

CONN. CAPACITY

2 - 3/4" φ HSB IN DBL SHR

$R_n = 2 \times 31.8^k / 0.75 = 85^k$ OK DCR = 0.12

10 I 25.4 GUSSET WELDED TO COL

x 4" x 2 SIDES (T & B) x 3/16" FILLET

$R_n = 4'' \times 2 \times 3 \times 1.39 / 0.75 = 44^K$ OK DCR = 0.24

BRACES OK

Degenkolb

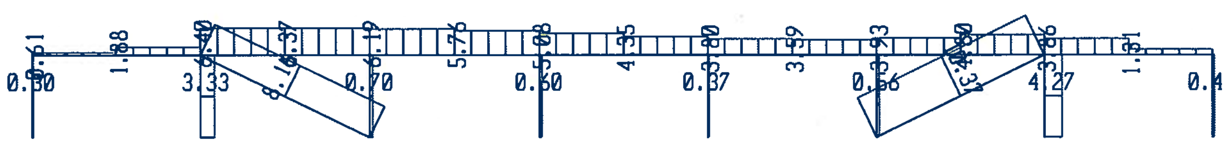
Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

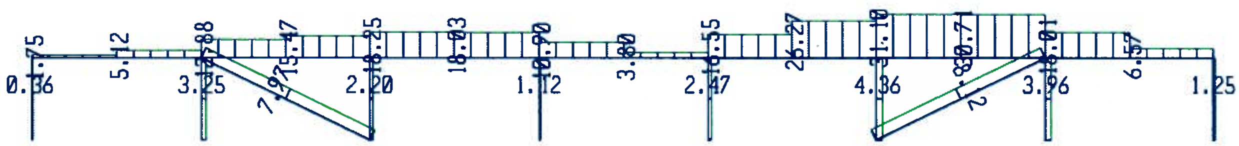
Subject:	Diaphragm Bracing Checks	Job Number: B3189012.	Date: 11.20.13
Job:	LLNL, B341 Increment I	By: AMN	Section:
		Checked By:	

Braces on Line 2a

Fye = 44 ksi

Section	Pcr (k)	Area (in2)	rx (in)	ry (in)	ro (in)	J (in4)	H	length (ft)	kl/ry	Fe (ksi)	Fcry (ksi)	Fcrz (ksi)	Fcr (ksi)
2L3X2X3/16 LLBB	3.6	1.83	0.961	0.739	1.49	0.0238	0.67	22	357.2	2.2	2.0	65.6	1.9


0.30
1.88
3.33
0.70
5.76
0.60
4.35
0.37
3.59
0.66
4.27
1.31
0.41



SAP2000 v15.2.1 - File:131115 B341 Increment I Model - Axial Force Diagram (BSE-1E-Y) - Kip, in, F Units

Evaluation of Diaphragms and Diaphragm Bracing

December 2013

Degenkolb

Degenkolb Engineers
235 Montgomery Street, Suite 500
San Francisco, CA 94104-2908
Phone 415.392.6952
Fax 415.981.3157

Subject: Diaphragm Checks
Job: LLNL B341 Increment I
Job Number: B3189012.00 Date: 12.17.13
By: AMN Section:
Checked By:

Level	Fpx/Fx
High Roof	1.00
Low Roof	1.22
Equip Loft	1.22
Mezz	1.71

Type	Strength (k/ft)
Metal Deck	0.81
6.5" Conc Slab	9.6

Diaphragm-Wall Connectin Strength		
Pre-cast Panels	3.3	k/ft
CIP High Roof	4.1	k/ft
CIP Mezz	7.6	k/ft

Section Cut	Level	Length (ft)	Earthquake	Vu (kips)	Amplification Factor	Diaphragm Demand (k/ft)	m-factor	Strength (k/ft)	Diaphragm DCR	Diaphragm-Wall Connection Strength	Connection DCR	
Line 1 High Roof Diaphragm	High Roof	180	BSE-1E-X	103	1.0	0.57	2.0	0.8	0.35	3.3	0.17	
Line 1 High Roof Diaphragm	High Roof	180	BSE-1E-Y	14	1.0	0.08	2.0	0.8	0.05	3.3	0.02	
Line 1 Mezz Diaphragm	Mezz	100	BSE-1E-X	44	1.7	0.76	2.0	9.6	0.04	3.3	0.23	
Line 1 Mezz Diaphragm	Mezz	100	BSE-1E-Y	22	1.7	0.38	2.0	9.6	0.02	3.3	0.12	
Line 1aa East Mezz Diaphragm	Mezz	100	BSE-1E-X	53	1.7	0.90	2.0	9.6	0.05	7.6	0.12	0.42
Line 1aa East Mezz Diaphragm	Mezz	100	BSE-1E-Y	23	1.7	0.39	2.0	9.6	0.02	7.6	0.05	0.13
Line 1aa West Mezz Diaphragm	Mezz	100	BSE-1E-X	134	1.7	2.28	2.0	9.6	0.12	7.6	0.30	
Line 1aa West Mezz Diaphragm	Mezz	100	BSE-1E-Y	35	1.7	0.60	2.0	9.6	0.03	7.6	0.08	
Line 1b East Mezz Diaphragm	Mezz	100	BSE-1E-X	77	1.7	1.31	2.0	9.6	0.07	7.6	0.17	0.67
Line 1b East Mezz Diaphragm	Mezz	100	BSE-1E-Y	28	1.7	0.48	2.0	9.6	0.03	7.6	0.06	0.17
Line 1b West Mezz Diaphragm	Mezz	100	BSE-1E-X	223	1.7	3.81	2.0	9.6	0.20	7.6	0.50	
Line 1b West Mezz Diaphragm	Mezz	100	BSE-1E-Y	48	1.7	0.83	2.0	9.6	0.04	7.6	0.11	
Line 2 A-E East High Roof Diaphragm	High Roof	80	BSE-1E-X	49	1.0	0.61	2.0	0.8	0.38	2.1	0.30	
Line 2 A-E East High Roof Diaphragm	High Roof	80	BSE-1E-Y	7	1.0	0.08	2.0	0.8	0.05	2.1	0.04	
Line 2 B-E West Equipment Loft Diaphragm	Equip Loft	60	BSE-1E-X	193	1.2	3.92	2.0	9.6	0.21	5.7	0.69	
Line 2 B-E West Equipment Loft Diaphragm	Equip Loft	60	BSE-1E-Y	48	1.2	0.97	2.0	9.6	0.05	5.7	0.17	
Line 2 B-E West High Roof Diaphragm	High Roof	60	BSE-1E-X	14	1.0	0.24	2.0	0.8	0.15	4.1	0.06	
Line 2 B-E West High Roof Diaphragm	High Roof	60	BSE-1E-Y	15	1.0	0.25	2.0	0.8	0.15	4.1	0.06	
Line 2 East High Roof Diaphragm	High Roof	180	BSE-1E-X	96	1.0	0.54	2.0	0.8	0.33	3.3	0.16	0.23
Line 2 East High Roof Diaphragm	High Roof	180	BSE-1E-Y	12	1.0	0.07	2.0	0.8	0.04	3.3	0.02	0.04
Line 2 West High Roof Diaphragm	High Roof	140	BSE-1E-X	31	1.0	0.22	2.0	0.8	0.14	3.3	0.07	
Line 2 West High Roof Diaphragm	High Roof	140	BSE-1E-Y	9	1.0	0.06	2.0	0.8	0.04	3.3	0.02	
Line 2a B-J Low Roof Diaphragm	Low Roof	140	BSE-1E-X	49	1.2	0.43	2.0	0.8	0.26	-	-	
Line 2a B-J Low Roof Diaphragm	Low Roof	140	BSE-1E-Y	21	1.2	0.18	2.0	0.8	0.11	-	-	
Line 2a High Roof Diaphragm	High Roof	140	BSE-1E-X	7	1.0	0.05	2.0	0.8	0.03	-	-	
Line 2a High Roof Diaphragm	High Roof	140	BSE-1E-Y	11	1.0	0.08	2.0	0.8	0.05	-	-	
Line 4 Low Roof Diaphragm	Low Roof	180	BSE-1E-X	88	1.2	0.60	2.0	0.8	0.37	3.3	0.18	
Line 4 Low Roof Diaphragm	Low Roof	180	BSE-1E-Y	13	1.2	0.08	2.0	0.8	0.05	3.3	0.03	

Degenkolb

Degenkolb Engineers
235 Montgomery Street, Suite 500
San Francisco, CA 94104-2906
Phone 415.392.6952
Fax 415.981.3157

Subject:	Diaphragm Checks		
Job:	LLNL B341 Increment I	Job Number: B3189012.00	Date: 12.17.13
		By: AMN	Section:
		Checked By:	

Level	Fpx/Fx
High Roof	1.00
Low Roof	1.22
Equip Loft	1.22
Mezz	1.71

Type	Strength (k/ft)
Metal Deck	0.81
6.5" Conc Slab	9.6

Diaphragm-Wall Connectin Strength		
Pre-cast Panels	3.3	k/ft
CIP High Roof	4.1	k/ft
CIP Mezz	7.6	k/ft

Section Cut	Level	Length (ft)	Earthquake	Vu (kips)	Amplification Factor	Diaphragm Demand (k/ft)	m-factor	Strength (k/ft)	Diaphragm DCR	Diaphragm-Wall Connection Strength	Connection DCR	
Line A High Roof Diaphragm	High Roof	50	BSE-1E-X	12	1.0	0.23	2.0	0.8	0.14	3.3	0.07	
Line A High Roof Diaphragm	High Roof	50	BSE-1E-Y	28	1.0	0.55	2.0	0.8	0.34	3.3	0.17	
Line A Low Roof Diaphragm	Low Roof	90	BSE-1E-X	18	1.2	0.25	2.0	0.8	0.15	3.3	0.07	
Line A Low Roof Diaphragm	Low Roof	90	BSE-1E-Y	63	1.2	0.86	2.0	0.8	0.53	3.3	0.26	
Line C North High Roof Diaphragm	High Roof	67	BSE-1E-X	14	1.0	0.21	2.0	0.8	0.13	4.1	0.05	
Line C North High Roof Diaphragm	High Roof	67	BSE-1E-Y	32	1.0	0.48	2.0	0.8	0.29	4.1	0.12	
Line E Mezz Diaphragm	Mezz	50	BSE-1E-X	48	1.7	1.63	2.0	9.6	0.09	7.6	0.21	
Line E Mezz Diaphragm	Mezz	50	BSE-1E-Y	63	1.7	2.15	2.0	9.6	0.11	7.6	0.28	
Line E South High Roof Diaphragm	High Roof	67	BSE-1E-X	10	1.0	0.15	2.0	0.8	0.09	4.1	0.04	
Line E South High Roof Diaphragm	High Roof	67	BSE-1E-Y	137	1.0	2.05	2.0	0.8	1.26	4.1	0.50	
Line F North Mezz Diaphragm	Mezz	50	BSE-1E-X	41	1.7	1.40	2.0	9.6	0.07	7.6	0.18	0.42
Line F North Mezz Diaphragm	Mezz	50	BSE-1E-Y	43	1.7	1.48	2.0	9.6	0.08	7.6	0.20	0.43
Line F South Mezz Diaphragm	Mezz	50	BSE-1E-X	52	1.7	1.77	2.0	9.6	0.09	7.6	0.23	
Line F South Mezz Diaphragm	Mezz	50	BSE-1E-Y	52	1.7	1.77	2.0	9.6	0.09	7.6	0.23	
Line F.7 North Mezz Diaphragm	Mezz	50	BSE-1E-X	49	1.7	1.67	2.0	9.6	0.09	7.6	0.22	0.50
Line F.7 North Mezz Diaphragm	Mezz	50	BSE-1E-Y	27	1.7	0.92	2.0	9.6	0.05	7.6	0.12	0.56
Line F.7 South Mezz Diaphragm	Mezz	50	BSE-1E-X	62	1.7	2.13	2.0	9.6	0.11	7.6	0.28	
Line F.7 South Mezz Diaphragm	Mezz	50	BSE-1E-Y	98	1.7	3.35	2.0	9.6	0.18	7.6	0.44	
Line H.3 North Mezz Diaphragm	Mezz	50	BSE-1E-X	60	1.7	2.04	2.0	9.6	0.11	7.6	0.27	0.55
Line H.3 North Mezz Diaphragm	Mezz	50	BSE-1E-Y	101	1.7	3.44	2.0	9.6	0.18	7.6	0.45	0.58
Line H.3 South Mezz Diaphragm	Mezz	50	BSE-1E-X	62	1.7	2.12	2.0	9.6	0.11	7.6	0.28	
Line H.3 South Mezz Diaphragm	Mezz	50	BSE-1E-Y	29	1.7	0.98	2.0	9.6	0.05	7.6	0.13	
Line J North Mezz Diaphragm	Mezz	50	BSE-1E-X	60	1.7	2.05	2.0	9.6	0.11	7.6	0.27	0.54
Line J North Mezz Diaphragm	Mezz	50	BSE-1E-Y	53	1.7	1.81	2.0	9.6	0.09	7.6	0.24	0.45
Line J South Mezz Diaphragm	Mezz	50	BSE-1E-X	60	1.7	2.04	2.0	9.6	0.11	7.6	0.27	
Line J South Mezz Diaphragm	Mezz	50	BSE-1E-Y	48	1.7	1.64	2.0	9.6	0.09	7.6	0.22	
Line K High Roof Diaphragm	High Roof	50	BSE-1E-X	9	1.0	0.18	2.0	0.8	0.11	3.3	0.05	
Line K High Roof Diaphragm	High Roof	50	BSE-1E-Y	131	1.0	2.62	2.0	0.8	1.62	3.3	0.80	
Line K Low Roof Diaphragm	Low Roof	90	BSE-1E-X	14	1.2	0.19	2.0	0.8	0.12	3.3	0.06	
Line K Low Roof Diaphragm	Low Roof	90	BSE-1E-Y	88	1.2	1.19	2.0	0.8	0.74	3.3	0.36	
Line K Mezz Diaphragm	Mezz	50	BSE-1E-X	58	1.7	1.98	2.0	9.6	0.10	3.3	0.60	
Line K Mezz Diaphragm	Mezz	50	BSE-1E-Y	64	1.7	2.18	2.0	9.6	0.11	3.3	0.66	

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject:	Metal Deck Roof Diaphragm Properties	**Job Number:**	B3189012.00	**Date:**	11.19.13
Job:	LLNL B341 Increment I	**By:**	AMN	**Section:**	
		Checked By:			

I. General Deck Information

h :	Deck Height (in.) =	1.50	
d :	Rib Spacing (in.) =	6	
t :	Base Metal Thickness (in.) =	0.0474	18 ga, Wide Rib
w :	Panel Width (in.) =	24	
L :	Panel Length (ft.) =	30	
P :	Weld Pattern =	3	
s_e :	Edge Weld Spacing, [w/(P-1)], (in.) =	12	
s_i :	Interior Stitch Spacing (in.) =	24	
w_c :	Corrugation dimension (in.) =	1.55	
e :	Corrugation dimension (in.) =	0.875	
f :	Corrugation dimension (in.) =	3.5	

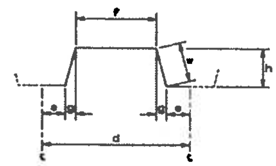


FIG 2.4-1 CORRUGATION DIMENSIONS

II. Connector Strength

Structural Fastener - Arc Spot Weld

d_w :	Weld Diameter (in.) =	0.5	
F_u :	Specified Minimum Steel Strength, Members (ksi) =	45	
Q_f :	Structural Connector Strength, $[2.2*t*F_u*(d_w-t)]$, (kips) =	2.12	
S_f :	Structural Connector Flexibility, $[1.15x10^{-3}/(t)^{0.5}]$, (in./kip) =	0.0053	

Sidelap Fastener - Button Punched (equivalent to #10 screw, per Verco manual)

Q_s :	Sidelap Connector Strength (kips) =	1.02	App IV
S_s :	Sidelap Connector Flexibility (in./kip) =	0.014	App IV

III. Diaphragm Strength

Edge Fasteners

α_1 :	End Dsitribution Factor, [App. IV] =	1.1
n_p :	No. of Purlins (excluding ends) =	3
α_2 :	Purlin Distribution Factor, [App. IV] =	1.1
n_e :	No. of Edge Connectors between Cross Supports, $[(L/(n_p+1))/s_e]$ =	7
S_{ue} :	Diaphragm Strength (Edge Limit), $[(2\alpha_1+n_p\alpha_2+n_e)Q_f/L]$ (Kip/ft) =	0.88

Interior Panels

L_v :	Purlin Spacing (ft.) =	7.50
λ :	$[1-h*L_v/(240*t^{0.5})]$ =	0.78
n_s :	No. of Stitch Connectors in Length, $[L/s_i]$:	15
α_s :	$[Q_s/Q_f]$ =	0.48
$\Sigma(x/w)^2$:	[App. IV] =	0.51
S_{ui} :	Diaphragm Strength (Interior Limit), $[2(\lambda-1)+n_s\alpha_s+\Sigma(x/w)^2*(2n_p+4)]*(Q_f/L)$, (kip/ft) =	0.84

End Members

N :	No. Fasters per Foot Along Ends =	1.00
B :	$[n_s\alpha_s+\Sigma(x/w)^2*(2n_p+4)]$ =	12.29
S_u :	Diaphragm Strength (Corner Limit), $[((N^2B^2/(L^2N^2+B^2))^{0.5})Qf]$, (kip/ft) =	0.81

Limiting Diaphragm Strength Value

S :	Diaphragm Strength (kip/ft) :	0.81

⇒ 810 plf COMPARES WELL TO $2\times V_{ALLOWABLE}$ = 2x455 plf = 910 plf
↑ FROM 1984 EVALUATION MEMO

IV. Stability Check

I :	Panel Moment of Inertia (in^4/ft width) =	0.305	
d_p :	Corrugation Pitch (in.) =	6	
s_d :	Developed Flute Width, $[2(e+w)+f]$, (in.) =	8.34	1.39
S_c :	Critical Buckling Load, $[(3.25*10^3/L_v^2)*(I^3t^3d_p/s_d)^{0.25}]$ (kips/ft) =	2.22	

IV. Diaphragm Stiffness

E :	Modulus of Elasticity (ksi) =	29000	
υ :	Poissons Ratio =	0.3	
C :	Slip Coefficient $[(Et/w)*S_f*((24L/(2\alpha_1+n_p\alpha_2+2n_sS_f/S_s))]$ =	12.83	
D:	Assume Wide Rib, [Table 3.31 pg. 3-4] =	13013	18 ga, every third valley
D_n :	Warping Constant [D/12L] =	36.15	
ϕ :	D_n Reduction [Table 3.3-2] =	0.80	
G' :	Diaphragm Stiffness (kips/in) =	30.30	
G:	Effective Modulus (ksi) =	639.35	
t' :	Effective Panel Thickness $[t*(G/11350)]$ (in.) =	0.002717	

Degenkolb

Subject:		Job Number:	Date:
Job:		By:	Section:
		Checked By:	Page of

MEZZ & EQUIPMENT LOFT DIAPH. CAP.

6 1/2" CONC. SLAB

$$v_c = 2\sqrt{2500 \times 1.5} \times 6.5'' \times 12'' / 1000$$

$$= 9.6^k/ft$$

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

DIAPHRAGM – WALL CONN.

WELD TO INSERT (1/4" FILLET)

$3'' \times 2 \times 4 \times 1.39/0.75 = \underline{\underline{45^K}}$

WELD TO BM (1/4" FILLET)

GAP BET. ANGLE & INSERT $= 1.125''$

IN-PLANE ECCENTRICITY $= 1.125'' + \frac{3-1.125}{2} = 2.06''$

$\angle\, 3 \times 3 \times 3/8 \times 0'-4''$

$A_{WELD} = 2 \times 1.875'' = 3.75\ in^2/in$

$S_{WELD} = 1.875 \times 2^2 \times 2 = 15\ in^3/in$

$\sigma_V = V/3.75 \qquad \sigma_b = 2.06'' \times V/15$

$\sigma_R = \sqrt{\left(\frac{V}{3.75}\right)^2 + \left(\frac{2.06V}{15}\right)^2} = 1.39/0.75 \times 4 = 7.4^{K/in}$

$\underline{\underline{V = 247^K}}$

CONCRETE ANCHOR

$\underline{\underline{V_n = 10.4^K}} \rightarrow$ SEE SPRD SHT.

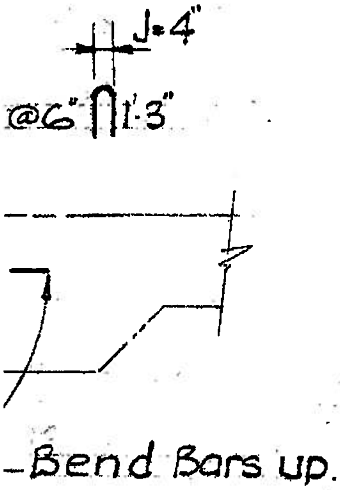
STRAPS

$V_n = 2 \times 3/16'' \times 1'' \times 0.6 \times 44^{ksi}$

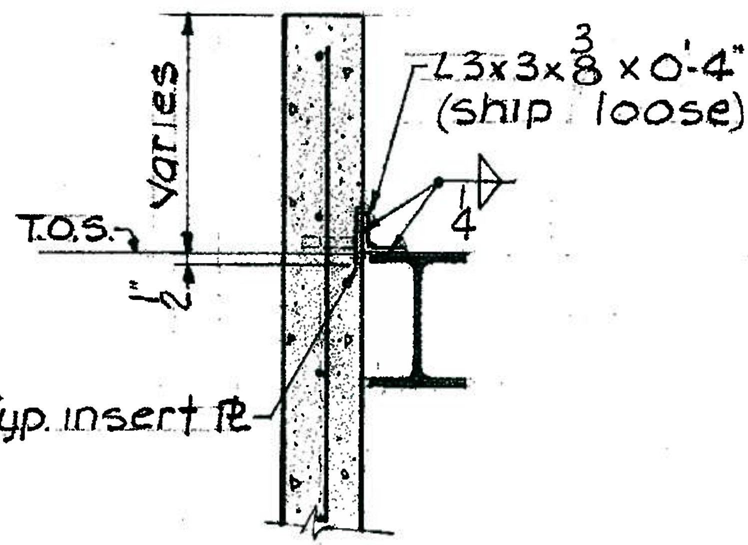
$= \underline{\underline{9.9^K}}$

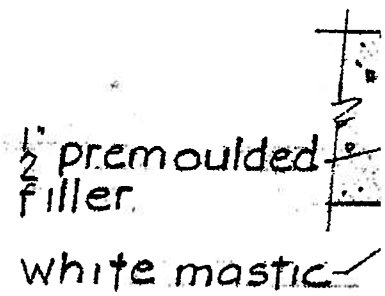
STRAP CONTROLS $V_n = \frac{9.9^K}{3'} \underline{\underline{= 3.3^{K/ft}}}$

>> DIAPHRAGM CAPACITY $= 0.8^{K/ft}$



@ 12





TYPI

DI

S

DOORS

SECTION F-11/13

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Precast Panel Connections	**Job Number:**	B31891012.00	**Date:**	12.4.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location: **Metal Deck to Wall Connection - Worst Case**
Condition: **In-plane Shear Connection: Diaphragm to panels**
Loading: **Shear parallel to panel edge**

Cast-In-Place Anchor Properties:

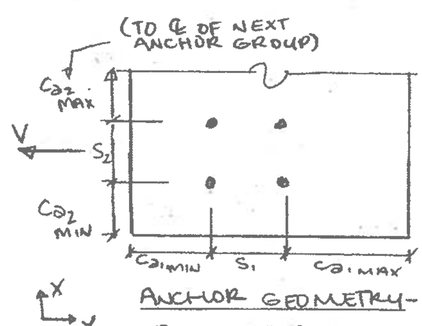
Material: F1554 Gr. 36
Diameter: 1/2" Φ
Nut Type: A563 Hex
Supplementary Reinforcement (between anchor and edge): None or <#4
Anchor Yield Stress, f_y: 44 ksi
Anchor Tensile Stress, f_u: 62 ksi
d_o = 0.500 in
Effective Embedment Depth of Anchor, h_{ef}: **2.63** in
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef} **N/A** in *(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support? **No**
Min Thickness of Steel Attachment : **N/A** in. *(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable: **0.00** in.^{2} *(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable: **0.00** in. *(Input '0.00' if not used)*
Embedded nut width, if applicable: **0.00** in

Concrete Data:

f'_c = **4000** psi
Concrete Type = Normal Weight
λ = 1.00 *ACI 8.6.1*
Concrete Performance: No Cracking at Service Loads
Concrete Depth, h_a: **6** in

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1: **3** in
Anchor Spacing in X-Direction, s_2: **0** in
Number of Anchor Rows in Y-Direction, n_y: **2**
Number of Anchor Rows in X-Direciton, n_x: **1**
Total Number of anchors in group, n: **2**
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}: **18.00** in
Y-Dir Edge Min. Edge Dist., c_{a1_min}: **18.0** in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}: **120.0** in
X-Dir Min. Edge Dist., c_{a2_min}: **10.8** in
Critical edge distance from ACI D.8.6, c_{ac}: 10.5 in
Dist from centroid of bolt group to applied tension load, e'_N*= **0.0** in
Diist from centroid of bolt group to applied shear load, e'_V* = **0.0** in
Can Shear Breakout Occur in Y-direction? **N**
Can Shear Breakout Occur in X-direction? **Y**



ANCHOR GEOMETRY - PLAN VIEW

**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point = **0** lbs
Max Shear at Conn Point = **1000** lbs
Direction of Shear Loading = Y-Dir
Required Incr. in Tension Demand =** **1.0**
Required Incr. in Shear Demand =** **1.0**

Tension Demand:
N_u = Tension at Anchor Group = **0** lbs

Shear Demand:
V_u = Shear at Anchor Group= **1000** lbs

****Required Demand Increases:**

Note: For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject:	Precast Panel Connections	**Job Number:**	B31891012.00	**Date:**	12.4.13
Job:	LLNL B341	**By:**		**Section:**	
		Checked By:		**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Metal Deck to Wall Connection - Worst Case**
Condition:	**In-plane Shear Connection: Diaphragm to panels**
Loading:	**Shear parallel to panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uta}$ *ACI Eqn (D-3)*

Φ =	0.75	
A_{se} =	0.142	in^2
f_{ut} =	62000	psi
ΦN_{sa} =	**13206**	**lbs**
N_u =	0	lbs
$N_u / \Phi N_{sa}$ =	$\boxed{0.00}$	

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ *ACI Eq (D-4)*

$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ *ACI Eq (D-5)*

$\Psi_{ec,N}$	1.00	*Eqn (D-9)*
$\Psi_{ed,N}$	1.00	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	1.25	*Section D.5.2.6*
$\Psi_{cp,N}$	1.00	*Does not apply to cast-in-place anchors*

Φ = 0.7

$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ OR $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$

k_c =	24	*ACI Eqn (D-7) or (D-8)*
N_b =	6456 lbs	
A_{Nco} =	$9 * h_{ef}^2$	*ACI Eqn (D-6)*
A_{Nco} =	62 in^2	*(Projected area for a single anchor without edge distance considered)*
A_{Nc} =	86 in^2	*(Proj. area for group of anchors with edge dist. considered; Not greater than $n * A_{Nco}$)*
		(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or N_{cbg} =	11144 lbs	*Strength of group of anchors*
0.75ΦNcb or 0.75ΦNcbg =	**5850 lbs**	
N_u =	**0 lbs**	
$N_u / \Phi N_{cbg}$ =	$\boxed{0.00}$	

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$		*ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$		*ACI Eqn (D-15)*
A_{brg} =	0.29 in^2	*From PCA Notes on ACI 318-05 Table 34-2*
Φ =	0.70	
N_p =	9312 lbs	
$\Psi_{c,P}$ =	1.40 in	
0.75ΦNpn =	**6844 lbs**	*Strength of a single anchor in group*
N_u =	**0 lbs**	*Demand of a single anchor in group*
$N_u / \Phi N_p$ =	$\boxed{0.00}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:**	12.4.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Metal Deck to Wall Connection - Worst Case**
Condition:	**In-plane Shear Connection: Diaphragm to panels**
Loading:	**Shear parallel to panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

$\Phi N_{sb} = \Phi^*\lambda^*160^*c_{a1}\sqrt{A_{brg}}\sqrt{f'c}$		*ACI Eqn (D-17)*
$\Phi N_{sbg} = \Phi^*(1+ s/(6^*c_{a1}))^*N_{sb}$		*ACI Eqn (D-18)*
$\Phi =$	0.70	
Deep embed. close to edge in Y-dir?	No	*"Yes" if $c_{a1} < 0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No	*"Yes" if $c_{a2} < 0.4h_{ef}$*
For anchor group , is $s_2 < 6^*c_{a1}$?	Yes	*For blowout of group in Y-direction*
For anchor group , is $s_1 < 6^*c_{a2}$?	Yes	*For blowout of group in X-direction*
Does failure mode apply in either dir?	No	*Failure mode applies if $c_{a\_min} < 0.4h_{ef}$ and if $s < 6^*c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	1.0	*Factor applies for single anchor if $c_{a2} < 3c_{a1}$ where $1.0 <= c_{a2}/c_{a1} <= 3.0$ and $c_{a1} = c_{a\_min}$*
$N_{sb} =$	N/A lbs	
$N_{sbg} =$	N/A lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A lbs**	
N_u =	**lbs**	
$N_u / \Phi N_{sb}$ =	**N/A**	

SHEAR CAPACITY: Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}

Steel Strength of Anchor in Shear: ACI D.6.1 *(for headed bolts)*

$\Phi V_{sa} = \Phi^*n^*0.6^*A_{se}^*f_{ut}$		*ACI Eqn (D-20)*
$\Phi =$	1	
$A_{se} =$	0.142	in^2
$f_{ut} =$	62000	psi
ΦV_{sa} =	**10565**	**lbs**
V_u =	**1000**	**lbs**
$V_u / \Phi V_{sa}$ =	**0.09**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location: **Metal Deck to Wall Connection - Worst Case**
Condition: **In-plane Shear Connection: Diaphragm to panels**
Loading: **Shear parallel to panel edge**

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} = \Phi * A_{Vc}/A_{Vco} * \Psi_{ed,V} * \Psi_{c,V} * V_b$		*ACI Eqn (D-21)*
$\Phi V_{cbg} = \Phi * A_{Vc}/A_{Vco} * \Psi_{ec,V} * \Psi_{ed,V} * \Psi_{c,V} * V_b$		*ACI Eqn (D-22)*
$V_b = \lambda * 7 * (\ell_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_{a1})^{1.5}$		*ACI Eqn (D-24)*
$A_{vc} =$ (Based on geometry)	$\leq n * A_{vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} = 4.5 * c_{a1}^2$		*Projected failure area in shear for a single anchor w/out considering edge dist*
$\Phi =$ 1.00		
$\ell_e =$ 2.6 in		*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V} =$ 1.4		*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

Assume all load acts on back anchor (check group):

$c_{a1} =$	21.00	in	
$V_b =$	41973	lbs	
$A_{Vc} =$	254	in^2	
$A_{Vco} =$	1985	in^2	
$\Psi_{ed,V} =$	0.80		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	1.00		*Eqn (D-26)*
$\Psi_{h,V} =$	2.29		*Eqn (D-29)*
$V_{cbg} =$	13800	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**10350**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.10}$		

Assume load is distributed equally between anchors (check front anchors): *(This case should not be considered if anchors in group are rigidly connected to support)*

$c_{a1} =$	18.00	in	
$V_b =$	33308	lbs	
$A_{Vc} =$	227	in^2	
$A_{Vco} =$	1458	in^2	
$\Psi_{ed,V} =$	0.82		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	1.00		*Eqn (D-26)*
$\Psi_{h,V} =$	2.12		*Eqn (D-29)*
$V_{cbg} =$	12593	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**9445**	**lbs**	
$V_u =$	500	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.05}$		

Shear Breakout in X-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):

$c_{a2} =$	10.75	in	
$V_b =$	15373	lbs	
$A_{Vc} =$	212	in^2	
$A_{Vco} =$	520	in^2	
$\Psi_{ed,V} =$	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	1.00		*Eqn (D-26)*
$\Psi_{h,V} =$	1.64		*Eqn (D-29)*
$V_{cbg} =$	28699	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**21524**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_x} =$	$\boxed{0.05}$		

Assume load is distributed equally between anchors (check front anchors): *(This case should not be considered if anchors in group are rigidly connected to support)*

$c_{a2} =$	10.75	in	
$V_b =$	15373	lbs	
$A_{Vc} =$	212	in^2	
$A_{Vco} =$	520	in^2	
$\Psi_{ed,V} =$	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V} =$	1.00		*Eqn (D-26)*
$\Psi_{h,V} =$	1.64		*Eqn (D-29)*
$V_{cbg} =$	28699	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**21524**	**lbs**	
$V_u =$	1000	lbs	
$V_u / \Phi V_{cbg\_y} =$	$\boxed{0.05}$		

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**Metal Deck to Wall Connection - Worst Case**
Condition:	**In-plane Shear Connection: Diaphragm to panels**
Loading:	**Shear parallel to panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi^* k_{cp}^* N_{cb}$ — *ACI Eqn (D-29); for single anchor*
$\Phi V_{cpg} = \Phi^* k_{cp}^* N_{cbg}$ — *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} =	11144	lbs
Φ =	1.00	
k_{cp} =	2.00	
0.75ΦVcp or 0.75ΦVcpg =	**16715**	**lbs**
V_u =	1000	lbs
$V_u / \Phi V_{cp}$ =	**0.06**	

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:

	Ductile Failure D/C	Non-Ductile Failure D/C*	
Steel Strength of Anchor in Tension:	0.00	0.00	
Concrete Breakout Strength of Anchor in Tension:	0.00	0.00	
Concrete Pullout Strength of Anchor in Tension:	0.00	0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:	N/A	N/A	
max	**0.00**	**0.00**	**< 1.0, OK**

Summary of Shear D/C Ratios:

	Ductile Failure D/C		Non-Ductile Failure D/C*	
Steel Strength of Anchor in Shear:	**0.09**		0.24	
Concrete Breakout Strength of Anchor in Shear:	**0.10**	Non-Ductile Failure Governs	0.24	
Concrete Pryout Strength of Anchor in Shear:	**0.06**		0.15	
max	**0.10**		**0.24**	**< 1.0, OK**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multiplied by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 2.0 in. — *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **Spacing OK**
Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. — *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

Check edge dist in direction of shear force: **Edge Dist OK**
Check edge dist perpendicular to shear force: **Edge Dist OK**

Degenkolb

Subject:

Job:

Job Number: Date:

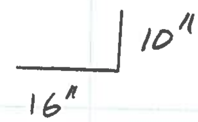
By: Section:

Checked By: Page of

DIAPHRAGM—WALL CONN. @ CIP WALLS

LINE 2 MEZZ

#5 @ 15" O.C.



$$\ell_d = \frac{40000\ psi}{25\sqrt{2500}}\, d_b = 32 d_b = 20''$$

SHEAR FRICTION

$$V_n = \frac{40^{ksi} \times 0.31\ in^2 \times 1.0 \times \frac{16-4}{20''}}{15''/12''} = 5.7^{k}/ft$$

$$\ell_{dh} = \frac{0.02 \times 40000}{\sqrt{2500}} \times 0.7 \times d_b = 11.2 d_b = 7''$$

SHEAR FRICTION

$$V_n = \frac{40^{ksi} \times 0.31\ in^2 \times 1.0 \times \frac{4''}{7''}}{15/12} = 5.7^{k}/ft \Leftarrow$$

CONTROLS

$$\underline{\underline{V_n = 5.7^{k}/ft}}$$

Degenkolb

Subject:

Job:

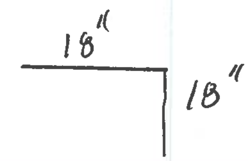
Job Number: Date:

By: Section:

Checked By: Page of

LINE E MEZZ

#4 @ 9" O.C.



$l_d = 32 d_b = 16''$

$l_{dh} = 11.2 d_b = 5.6''$

SHEAR FRICTION

$$v_n = \frac{\frac{18''-4''}{16''} \times 40^{ksi} \times 0.2 in^2 \times 1.0}{9/12} = 9.3 \, ^k/ft$$

$$v_n = \frac{\frac{4''}{5.6''} \times 40^{ksi} \times 0.2 in^2 \times 1.0}{9/12} = 7.6 \, ^k/ft$$

$$\underline{\underline{v_n = 7.6 \, ^k/ft}}$$

Degenkolb

Subject: | Job Number: | Date:
Job: | By: | Section:
Checked By: | Page of

LINE E & C - HIGH ROOF

3/4"ϕ A.B. @ 2'-0" O.C. w/ CONT. L4x3x3/8

ANCHOR STRENGTH GOVERNS, SEE SPRD SHT.

$V_n = 12.4^k$

$v_n = 12.4^k/2' = \underline{\underline{6.2^k/ft}}$

LINE 2 - HIGH ROOF

3/4"ϕ A.B. @ 3' O.C. w/ CONT. L7x4x3/8

$V_n = 12.4^k$, SAME AS PREV.

$v_n = 12.4^k/3' = \underline{\underline{4.1^k/ft}}$

Degenkolb

Subject:
Job:

Job Number: Date:
By: Section:
Checked By: Page of

LINE 19a & 16 MEZZ

CONST. JT. IN WALL @ BOT OF BMS
SHEAR FRICTION PROVIDED BY WALL VERT. STL

#4 @ 18" O.C., E.F.

→ #4 @ 9" O.C.

SAME AS LINE E @ MEZZ

$V_n = 7.6$ k/ft

EAST-WEST WALLS @ MEZZ.

VERT WALL BARS HOOKED INTO TOP OF
6½" SLAB

2-#4 @ 18" O.C.

∴ $V_n = 7.6$ k/ft

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:** 12.4.13
Job: LLNL B341	**By:**	**Section:**
	Checked By:	**Page/of:**

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**High Roof Cast in place anchor bolt**
Condition:	**In-plane Shear Connection: diaphragm to CIP walls**
Loading:	**Shear parallel to panel edge**

Cast-In-Place Anchor Properties:

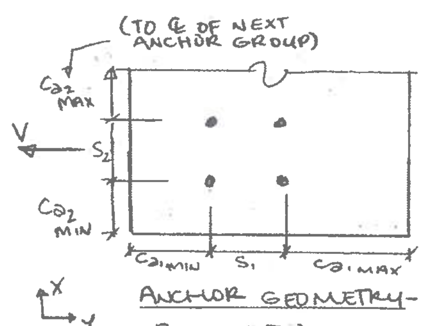
Material:	F1554 Gr. 36		
Diameter:	3/4" Φ		
Nut Type:	A563 Hex		
Supplementary Reinforcement (between anchor and edge):	None or <#4		
Anchor Yield Stress, f_y:	44	ksi	
Anchor Tensile Stress, f_u:	62	ksi	
d_o =	0.750	in	
Effective Embedment Depth of Anchor, h_{ef}:	**5.50**	in	
Embed from ACI D.5.2.3 for eqns (D-4)-(D-11), h'_{ef}	**N/A**	in	*(for anchors close to three or more edges)*
Are anchors in group rigidly connected to support?	**No**		
Min Thickness of Steel Attachment :	**N/A**	in.	*(ACI Sec. D.6.2.3, max of 3/8" and 0.5*do)*
Embedded washer plate area, if applicable:	**0.00**	in.^{2}	*(Input '0.00' if not used)*
Embedded washer plate thickness, if applicable:	**0.00**	in.	*(Input '0.00' if not used)*
Embedded nut width, if applicable:	**0.00**	in	

Concrete Data:

f'_c =	**4000**	psi	
Concrete Type =	Normal Weight		
λ =	1.00		*ACI 8.6.1*
Concrete Performance:	No Cracking at Service Loads		
Concrete Depth, h_a:	**6**	in	

Anchor Geometry:

Anchor Spacing in Y-Direction, s_1:	**0**	in
Anchor Spacing in X-Direction, s_2:	**0**	in
Number of Anchor Rows in Y-Direction, n_y:	**1**	
Number of Anchor Rows in X-Direciton, n_x:	**1**	
Total Number of anchors in group, n:	1	
Y-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a1_max}:	**12.00**	in
Y-Dir Edge Min. Edge Dist., c_{a1_min}:	**12.0**	in
X-Dir: 1/2 Dist to Next Anchor Group OR Max. Edge Dist., c_{a2_max}:	**120.0**	in
X-Dir Min. Edge Dist., c_{a2_min}:	**18.0**	in
Critical edge distance from ACI D.8.6, c_{ac}:	22	in
Dist from centroid of bolt group to applied tension load, e'_N*=	**0.0**	in
Diist from centroid of bolt group to applied shear load, e'_V* =	**0.0**	in
Can Shear Breakout Occur in Y-direction?	**N**	
Can Shear Breakout Occur in X-direction?	**Y**	



ANCHOR GEOMETRY - PLAN VIEW

**Note that e'_N and e'_V do not account for additional T or V on anchors due to eccentric application of load. They are only used to calculate the ψ factors for shear and tension breakout.*

Anchor Demands:

Do anchors resist seismic demands in a moderate or high seismic region? **Y** *(If "Y", include 0.75 decrease on capacity for concrete failure modes per D.3.3.3)*

Max Tension at Conn Point =	**0**	lbs
Max Shear at Conn Point =	**1000**	lbs
Direction of Shear Loading =	Y-Dir	
Required Incr. in Tension Demand =**	**1.0**	
Required Incr. in Shear Demand =**	**1.0**	

Tension Demand:

N_u = Tension at Anchor Group = **0** lbs

Shear Demand:

V_u = Shear at Anchor Group= **1000** lbs

****Required Demand Increases:**

Note: For anchorage of nonstructural components, increase demands by a factor of 1.3 per ASCE 7-05 Ch13.4.2a.
Per CBC 2010 Section 1615A.1.14 this increase is no longer required for OSHPD jobs.

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:**	12.4.13
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**High Roof Cast in place anchor bolt**
Condition:	**In-plane Shear Connection: diaphragm to CIP walls**
Loading:	**Shear parallel to panel edge**

Anchor Capacities:

Note: *Capacities associated with concrete failure modes are multiplied by 0.75 per ACI 318-08 D.3.3.3 for structures in Seismic Design Categories D, E, F.*

TENSION CAPACITY: Lowest of ΦN_{sa}, ΦN_{cb}, ΦN_{pn}, ΦN_{sb}

Steel Strength of Anchor in Tension: ACI D.5.1

$\Phi N_{sa} = \Phi * n * A_{se} * f_{uts}$ — *ACI Eqn (D-3)*

Φ =	0.75	
A_{se} =	0.334	in^2
f_{ut} =	62000	psi
ΦN_{sa} =	**15531**	**lbs**
N_u =	0	lbs
$N_u / \Phi N_{sa}$ =	$\boxed{0.00}$	

Concrete Breakout Strength of Anchor in Tension: ACI D.5.2

$\Phi N_{cb} = \Phi * A_{nc}/A_{nco} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-4)*

$\Phi N_{cbg} = \Phi * A_{nc}/A_{nco} * \Psi_{ec,N} * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$ — *ACI Eq (D-5)*

$\Psi_{ec,N}$	1.00	*Eqn (D-9)*
$\Psi_{ed,N}$	1.00	*Eqns (D-10 and D-11)*
$\Psi_{c,N}$	1.25	*Section D.5.2.6*
$\Psi_{cp,N}$	1.00	*Does not apply to cast-in-place anchors*

Φ = 0.7

$N_b = \lambda * k_c * \sqrt{f'_c} * (h_{ef})^{1.5}$ **OR** $N_b = \lambda * 16 * \sqrt{f'_c} * (h_{ef})^{5/3}$ if $(11 < h_{ef} < 25)$ — *ACI Eqn (D-7) or (D-8)*

k_c =	24	
N_b =	19579 lbs	
A_{Nco} =	**$9 * h_{ef}^2$**	*ACI Eqn (D-6)*
A_{Nco} =	272 in^2	*(Projected area for a single anchor without edge distance considered)*
A_{Nc} =	272 in^2	*(Proj. area for group of anchors with edge dist. considered; Not greater than $n*A_{Nco}$)*
		(Where washers are used, projected area is calculated per ACI 318-05 Section D.5.2.8.)
N_{cb} or N_{cbg} =	24473 lbs	*Strength of group of anchors*
0.75ΦNcb or 0.75ΦNcbg =	**12849 lbs**	
N_u =	**0 lbs**	
$N_u / \Phi N_{cbg}$ =	$\boxed{0.00}$	

Concrete Pullout Strength of Anchor in Tension: ACI D.5.3

$\Phi N_{pn} = \Phi * \Psi_{c,P} * N_p$		*ACI Eqn (D-14)*
$N_p = 8 * A_{brg} * f'_c$		*ACI Eqn (D-15)*
A_{brg} =	0.65 in^2	*From PCA Notes on ACI 318-05 Table 34-2*
Φ =	0.70	
N_p =	20928 lbs	
$\Psi_{c,P}$ =	1.40 in	
0.75ΦNpn =	**15382 lbs**	*Strength of a single anchor in group*
N_u =	**0 lbs**	*Demand of a single anchor in group*
$N_u / \Phi N_p$ =	$\boxed{0.00}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:** 12.4.13	
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**High Roof Cast in place anchor bolt**
Condition:	**In-plane Shear Connection: diaphragm to CIP walls**
Loading:	**Shear parallel to panel edge**

TENSION CAPACITY, cont.

Concrete Side-Face Blowout Strength of a Headed Anchor in Tension: ACI D.5.4

$\Phi N_{sb} = \Phi^*\lambda^*160^*c_{a1}\sqrt{A_{brg}}\sqrt{f'c}$		*ACI Eqn (D-17)*
$\Phi N_{sbg} = \Phi^*(1+ s/(6^*c_{a1})^*N_{sb}$		*ACI Eqn (D-18)*
$\Phi =$	0.70	
Deep embed. close to edge in Y-dir?	No	*"Yes" if $c_{a1}<0.4h_{ef}$*
Deep embed. close to edge in X-dir?	No	*"Yes" if $c_{a2}<0.4h_{ef}$*
For anchor group , is $s_2<6^*c_{a1}$?	Yes	*For blowout of group in Y-direction*
For anchor group , is $s_1<6^*c_{a2}$?	Yes	*For blowout of group in X-direction*
Does failure mode apply in either dir?	No	*Failure mode applies if $c_{a\_min}<0.4h_{ef}$ and if $s<6^*c_a$ (for group of anchors)*
Edge distance factor for N_{sb}:	0.6	*Factor applies for single anchor if $c_{a2} < 3c_{a1}$ where $1.0<= c_{a2}/c_{a1} <=3.0$ and $c_{a1} = c_{a\_min}$*
$N_{sb} =$	N/A lbs	
$N_{sbg} =$	N/A lbs	*(where N_{sb} doesn not include edge distance factor)*
0.75ΦNsb or 0.75ΦNcbg =	**N/A lbs**	
$N_u =$	lbs	
$N_u / \Phi N_{sb} =$	$\boxed{\text{N/A}}$	

SHEAR CAPACITY: Lowest of ΦV_{sa}, ΦV_{cg}, ΦV_{cp}

Steel Strength of Anchor in Shear: ACI D.6.1 *(for headed bolts)*

$\Phi V_{sa} = \Phi^*n^*0.6^*A_{se}^*f_{ut}$ *ACI Eqn (D-20)*

$\Phi =$	1	
$A_{se} =$	0.334	in^2
$f_{ut} =$	62000	psi
$\Phi V_{sa} =$	12425	lbs
$V_u =$	1000	lbs
$V_u / \Phi V_{sa} =$	$\boxed{0.08}$	

Degenkolb

Degenkolb Engineers
1300 Clay St, 9th Floor
Oakland, CA 94612-2047
Phone: 510.272.9040
Fax: 510.272.9526

Subject: Precast Panel Connections	**Job Number:** B31891012.00	**Date:** 12.4.13	
Job: LLNL B341	**By:**	**Section:**	
	Checked By:	**Page/of:**	

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location: High Roof Cast in place anchor bolt
Condition: In-plane Shear Connection: diaphragm to CIP walls
Loading: Shear parallel to panel edge

Concrete Breakout Strength of Anchor in Shear: ACI D.6.2

Note: *Assume all load acts on back anchor, and check anchor group. See ACI 318-05 Fig. RD.6.2.1(b).*

$\Phi V_{cb} = \Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$		*ACI Eqn (D-21)*
$\Phi V_{cbg} = \Phi^* A_{Vc}/A_{Vco}{}^*\Psi_{ec,V}{}^*\Psi_{ed,V}{}^*\Psi_{c,V}{}^*V_b$		*ACI Eqn (D-22)*
$V_b = \lambda^*7^*(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$		*ACI Eqn (D-24)*
A_{vc} = (Based on geometry)	$\leq n^*A_{vco}$	*ACI Eqn (D-23): Proj. failure area in shear for group of anchors considering edge dist*
$A_{Vco} = 4.5^*c_{a1}{}^2$		*Projected failure area in shear for a single anchor w/out considering edge dist*
Φ = 1.00		
l_e = 5.5 in		*Load bearing length of anchor for shear, ACI Section D.6.2.2*
$\Psi_{c,V}$ = 1.4		*ACI Section D.6.2.7*

Shear Breakout in Y-Direction:

Assume all load acts on back anchor (check group):				Assume load is distributed equally between anchors (check front anchors):			
c_{a1} =	12.00	in		c_{a1} =	12.00	in	
V_b =	23741	lbs		V_b =	23741	lbs	
A_{Vc} =	216	in^2		A_{Vc} =	216	in^2	
A_{Vco} =	648	in^2		A_{Vco} =	648	in^2	
$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*	$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	1.00		*Eqn (D-26)*	$\Psi_{ec,V}$ =	1.00		*Eqn (D-26)*
$\Psi_{h,V}$ =	1.73		*Eqn (D-29)*	$\Psi_{h,V}$ =	1.73		*Eqn (D-29)*
V_{cbg} =	19189	lbs		V_{cbg} =	19189	lbs	
0.75Φ(Vcb_y or Vcbg_y)=	**14392**	**lbs**		**0.75Φ(Vcb_y or Vcbg_y)=**	**14392**	**lbs**	
V_u =	1000	lbs		V_u =	1000	lbs	
$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.07}$			$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.07}$		

(This case should not be considered if anchors in group are rigidly connected to support)

Shear Breakout in X-Direction:

(Capacities increased by 2 for loading parallel to edge per ACI Section D6.2.1)

Assume all load acts on back anchor (check group):				Assume load is distributed equally between anchors (check front anchors):			
c_{a2} =	8.00	in		c_{a2} =	8.00	in	
V_b =	12923	lbs		V_b =	12923	lbs	
A_{Vc} =	144	in^2		A_{Vc} =	144	in^2	
A_{Vco} =	288	in^2		A_{Vco} =	288	in^2	
$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*	$\Psi_{ed,V}$ =	1.00		*Eqn (D-27) or (D-28)*
$\Psi_{ec,V}$ =	1.00		*Eqn (D-26)*	$\Psi_{ec,V}$ =	1.00		*Eqn (D-26)*
$\Psi_{h,V}$ =	1.41		*Eqn (D-29)*	$\Psi_{h,V}$ =	1.41		*Eqn (D-29)*
V_{cbg} =	25586	lbs		V_{cbg} =	25586	lbs	
0.75Φ(Vcb_x or Vcbg_x)=	**19189**	**lbs**		**0.75Φ(Vcb_x or Vcbg_x)=**	**19189**	**lbs**	
V_u =	1000	lbs		V_u =	1000	lbs	
$V_u / \Phi V_{cbg\_x}$ =	$\boxed{0.05}$			$V_u / \Phi V_{cbg\_y}$ =	$\boxed{0.05}$		

(This case should not be considered if anchors in group are rigidly connected to support)

Anchorage to Concrete per ACI 318-08 Appendix D

Anchorage Condition:

Location:	**High Roof Cast in place anchor bolt**
Condition:	**In-plane Shear Connection: diaphragm to CIP walls**
Loading:	**Shear parallel to panel edge**

SHEAR CAPACITY, cont.

Concrete Pryout Strength of Anchor in Shear: ACI D.6.3

$\Phi V_{cp} = \Phi * k_{cp} * N_{cb}$ — *ACI Eqn (D-29); for single anchor*

$\Phi V_{cpg} = \Phi * k_{cp} * N_{cbg}$ — *ACI Eqn (D-30); for group of anchors*

N_{cb} or N_{cbg} = 24473 lbs

Φ = 1.00

k_{cp} = 2.00

0.75ΦVcp or 0.75ΦVcpg = 36710 lbs

V_u = 1000 lbs

$V_u / \Phi V_{cp}$ = $\boxed{0.03}$

Design Summary and Combined Loading Checks:

Summary of Tension D/C Ratios:

		Ductile Failure D/C		Non-Ductile Failure D/C*	
Steel Strength of Anchor in Tension:		0.00		0.00	
Concrete Breakout Strength of Anchor in Tension:		0.00		0.00	
Concrete Pullout Strength of Anchor in Tension:		0.00		0.00	
Concrete Side-Face Blowout Strength of Anchor in Tension:		N/A		N/A	
	max	0.00	max	0.00	< 1.0, OK

Summary of Shear D/C Ratios:

		Ductile Failure D/C			Non-Ductile Failure D/C*
Steel Strength of Anchor in Shear:		**0.08**	Ductile Failure Governs		0.20
Concrete Breakout Strength of Anchor in Shear:		**0.07**			0.17
Concrete Pryout Strength of Anchor in Shear:		**0.03**			0.07
	max	**0.08**	< 1.0, OK	max	**0.20**

**Note: Where a ductile failure does not govern, ACI 318-08 Section D.3.3.6 requires that the design strength, determined in accordance with Section D.3.3.3, must be multiplied by a factor of 0.4.*

Combined Loading Per ACI D.7:

Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Non-Ductile Failure:

Combined D/C => **Combined Effects need not be considered when either Tension or Shear D/C ≤ 0.20**

D/C = **N/A**

Check Minimum Edge Distances and Spacings Per ACI D.8:

Minimum Center to Center Spacing = 3.0 in. — *ACI D.8.1, $4d_o$ for untorqued cast-in anchor*

Check spacing in X-Direction: **N/A**

Check spacing in Y-Direction: **N/A**

Minimum Edge Distance = 2 in. — *ACI D.8.2, Edge distance requirements based on cover requirements of ACI 318-05 Section 7.7.*

Check edge dist in direction of shear force: **Edge Dist OK**

Check edge dist perpendicular to shear force: **Edge Dist OK**

Degenkolb

Subject:
Job:
Job Number: Date:
By: Section:
Checked By: Page of

CHECK COLLECTORS TO CONC. WALLS @ LOW ROOF ON LINES C & E

DEMAND @ E = 195^k
DEMAND @ C = 125^k
FROM ANALYSIS, SEE SAP OUTPUT NEXT PG.

FOR FORCE CONTROLLED DIVIDE BY $C_1 C_2 = 1.4$

$$P_{u_E} = 195^k / 1.4 = 140^k$$

$$P_{u_C} = 125^k / 1.4 = 90^k$$

THERE IS NO SIGNIFICANT TENSILE LOAD PATH @ THESE LOCATIONS, AT BEST 1- #5 FROM SLAB INTO WALL

$$T_n = 0.31 \times 40^k \times \frac{16''}{32 \times 0.625''} = 10^k$$

DCR >> 1.0

NO GOOD

TABLE: Section Cut Forces - Analysis				
SectionCut	OutputCase	F1	F2	F3
Text	Text	Kip	Kip	Kip
Line C/2 Low Roof Collector	BSE-1E-X	26.143	14.921	3.672
Line C/2 Low Roof Collector	BSE-1E-Y	16.399	125.21	2.667
Line E/2 Low Roof Collector	BSE-1E-X	71.533	15.745	3.617
Line E/2 Low Roof Collector	BSE-1E-Y	57.36	194.5	1.868

Degenkolb

Subject:		Job Number:	Date:
Job:		By:	Section:
		Checked By:	Page of

CHECK COLLECTOR ALONG LINE 2 @ EQUIP. LOFT

$T_u = 124^k$ FROM SAP MODEL

FORCE CONTROLLED ∴ DIVIDE BY $C_1C_2 = 1.4$

$T_u = 124^k/1.4 = 89^k$

W12x40 BM

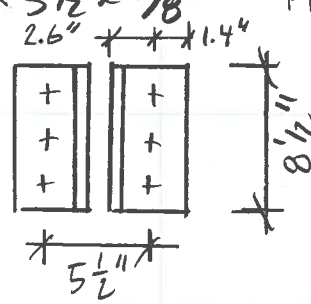
STD B-3 CONN: 2L's w/ 3-3/4"Ø BOLTS @ BM & 6-3/4"Ø BOLTS @ COL

CHECK BOLTS @ BM

$R_n = (3)(31.8^k/\phi{=}0.75) = 127^k$ OK DBL SHEAR

CHECK PRYING ON COL CONN

2L 4×3½×3/8 FROM AISC 1958 – SEE FOLLOWING PAGES



$F_u = 62^{ksi}$

$B = 39.7^k$

$p = 3"$ $d' = 13/16"$

$b = 2.41"$ $b' = 2.41" - 13/32" = 2"$

$a = 1.4"$ $a' = 1.4" + 13/32" = 1.81"$

$\rho = \frac{b'}{a'} = 1.1$

BOLT PRYING FORCE → $q = B\left[\delta\alpha\rho\left(t/t_c\right)^2\right]$ $t = 3/8"$

$t_c = \sqrt{\dfrac{4.44 \times 39.7 \times 2"}{3" \times 62^{ksi}}} = 1.37"$

$\delta = 1 - \dfrac{d'}{p} = 1 - \dfrac{13/16}{3} = 0.73$

Degenkolb

Subject:
Job:
Job Number:
Date:
By:
Section:
Checked By:
Page of

$T_{AVAIL} = B\,Q$ AVAILABLE STRENGTH OF BOLT INCLUDING TENSION + PRYING

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$$

$$= \frac{1}{0.73(1+1.1)}\left[\left(\frac{1.37}{0.375}\right)^2 - 1\right] = 8$$

$\therefore$

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta) = \left(\frac{0.375}{1.37}\right)^2 (1+0.73) = 0.13$$

$$T_{AVAIL} = 0.13 \times 39.7^k = 5.1^k$$

$$T_n = 6 \times 5.1^k = 30.6^k$$

$$DCR = 89^k / 30.6^k = 2.9$$ NO GOOD

OF BAYS TO STRENGTHEN @ WALL

WALL CONN IS GOOD FOR $5.7^k/ft$

$90^k / 5.7^k/ft = 16'$

$\therefore$ STRENGTHEN 2 BAYS. ℄ WALL

& 4 BAYS SOUTH

$\therefore$ GRIDS D – H

TABLE: Section Cut Forces - Analysis				
SectionCut	OutputCase	F1	F2	F3
Text	Text	Kip	Kip	Kip
Line 2/E Low Roof Collector	BSE-1E-X	124.154	18.952	1.04
Line 2/E Low Roof Collector	BSE-1E-Y	98.923	60.849	0.482

RIVETS ¾″

STANDARD BEAM CONNECTIONS

AMERICAN STANDARD BEAMS

WEIGHTS AND MINIMUM SPANS FOR ALLOWABLE UNIFORM LOADS

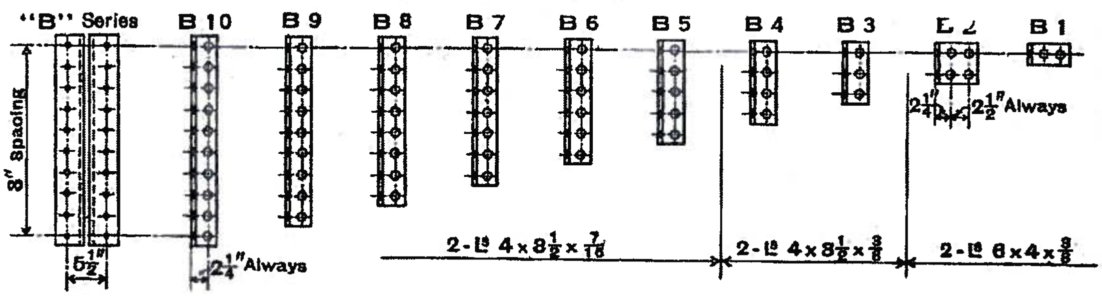
Notes on page 152 apply in general.
For Channels use same standard connection as for American Standard Beam of same depth.

Section		"B" Connection			"K" Connection			"KK" Connection		
Depth In.	Wt. Lb.	Symbol	Wt. Lb.	Min. Span Feet	Symbol	Wt. Lb.	Min. Span Feet	Sym-bol	Wt. Lb.	Min. Span Feet
24	120 105.9 100 90 79.9	B 6	34	21.1 19.7 16.6 15.6 14.6				KK 6	55	12.6 11.8 9.9 9.3 8.8
20	95 85 75 65.4	B 5	28	16.1 15.1 12.7 11.8				KK 5	46	10.1 9.4 7.9 7.4
18	70 54.7	B 4	20	12.8 11.1				KK 4	33	6.4 5.6
15	50 42.9	B 4	20	8.1 8.0†	K 4	28	7.4	KK 4	33	4.0 4.9†

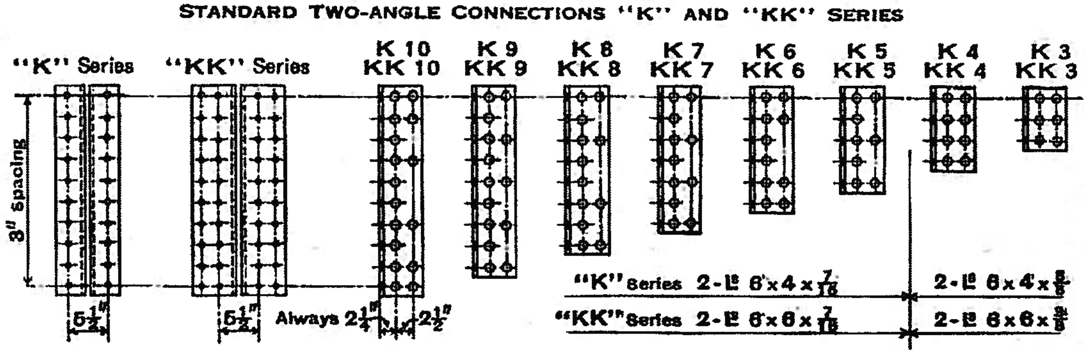
Section		"B" Connection			"K" Connection			"KK" Connection		
Depth In.	Wt. Lb.	Symbol	Wt. Lb.	Min. Span Feet	Symbol	Wt. Lb.	Min. Span Feet	Sym-bol	Wt. Lb.	Min. Span Feet
12	50 40.8 35 31.8	B 3	14	8.4 7.5 6.5† 7.6†	K 3	20	6.3 6.0	KK 3	24	4.2 4.1† 3.8† 4.4†
10	35 25.4	B 2	13	7.3 6.1						
8	23 18.4	B 2	13	4.0 3.6						
7	20 15.3	B 1	7	6.0 5.2						
6	17.25 12.5	B 1	7	4.4 3.7						
5	14.75 10	B 1	7	3.0 2.5						

†These spans are governed by web bearing or web shear.

STANDARD TWO-ANGLE CONNECTIONS "B" SERIES



STANDARD TWO-ANGLE CONNECTIONS "K" AND "KK" SERIES



Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

CHECK DIAPHRAGM CHORDS @ LOW & HIGH ROOF

HIGH ROOF

DIAPHRAGM CAPACITY = 0.8 k/ft

MAX SPAN = 100'

DEPTH = 50'

$$w = \frac{0.8^{k}/ft \times 50' \times 2}{100'} = 0.8^{k}/ft$$

$$M = wl^2/8 = (0.8^{k}/ft)(100')^2/8$$

$$= 1000^{k-ft}$$

$$P_{CHORD} = 1000^{k-ft}/50' = 20^{k}$$

10 B21 w/ 2L 5x3x3/8 CONN.

$$T_n = 4 \times 5.1^{k} = 20.4^{k} \cong 20^{k} \quad \underline{\underline{OK}}$$

STD "B CONN." SEE MECH. ROOM CONN. CALC.

LOW ROOF

OK BY COMPARISON TO HIGH ROOF

Subject:	Diaphragm Bracing Checks	Job Number: B3189012.00	Date: 11.20.13
Job:	LLNL, B341 Increment I	By: AMN	Section:
		Checked By:	

Brace Properties

Fye = 44 ksi

Section	Pcr (k)	Area (in2)	rx (in)	ry (in)	ro (in)	J (in4)	H	length (ft)	kl/ry	Fe (ksi)	Fcry (ksi)	Fcrz (ksi)	Fcr (ksi)
2L4X3X1/4LLBB	65.3	3.38	1.28	1.15	2.03	0.0772	0.71	10	104.3	26.3	21.8	62.1	19.3
2L5X3-1/2X5/16LLBB	121.9	5.12	1.61	1.32	2.51	0.1766	0.68	9	81.8	42.8	28.6	61.3	23.8

Bracing Member Checks

		Buckling Capacity	BSE-1E-X	BSE-1-Y		Brace Buckling		
SAP Element	Frame Section	Pcr (k)	Pmax	Pmax	m-factor	DCRmax	Connection Capacity (k)	Connection DCR
168	2L4X3X1/4LLBB	65	39	21	5.0	0.12	56.0	0.69
169	2L4X3X1/4LLBB	65	11	22	5.0	0.07	76.0	0.28
170	2L4X3X1/4LLBB	65	39	24	5.0	0.12	56.0	0.69
171	2L4X3X1/4LLBB	65	13	28	5.0	0.09	76.0	0.37
172	2L4X3X1/4LLBB	65	32	19	5.0	0.10	56.0	0.58
173	2L4X3X1/4LLBB	65	6	17	5.0	0.05	76.0	0.22
174	2L4X3X1/4LLBB	65	31	14	5.0	0.10	56.0	0.56
175	2L4X3X1/4LLBB	65	7	21	5.0	0.06	76.0	0.28
180	2L4X3X1/4LLBB	65	16	30	5.0	0.09	76.0	0.39
181	2L4X3X1/4LLBB	65	33	56	5.0	0.17	56.0	1.00
182	2L4X3X1/4LLBB	65	18	40	5.0	0.12	76.0	0.52
183	2L4X3X1/4LLBB	65	19	81	5.0	0.25	56.0	1.44
184	2L5X3-1/2X5/16LLBB	122	30	22	5.0	0.05	76.0	0.40
185	2L5X3-1/2X5/16LLBB	122	28	31	5.0	0.05	76.0	0.40
186	2L5X3-1/2X5/16LLBB	122	34	28	5.0	0.06	76.0	0.45
187	2L5X3-1/2X5/16LLBB	122	41	16	5.0	0.07	76.0	0.54
206	2L4X3X1/4LLBB	65	10	13	5.0	0.04	54.0	0.25
207	2L4X3X1/4LLBB	65	11	14	5.0	0.04	54.0	0.27
208	2L4X3X1/4LLBB	65	9	13	5.0	0.04	54.0	0.23
209	2L4X3X1/4LLBB	65	13	15	5.0	0.05	54.0	0.28

Degenkolb

Subject:
Job:
Job Number: Date:
By: Section:
Checked By: Page of

DIAPHRAGM BRACING CONN. STRENGTHS

BOLTS $= 3/4''\phi$ A325

TYP. CONN – SEE PAGES

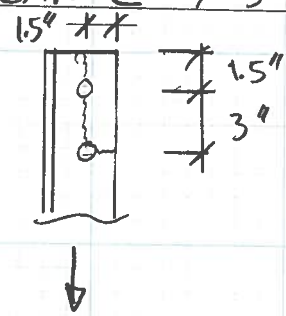
BOLT CAPACITY $= 4 \times 15.9^k / \phi = 0.75 = \underline{\underline{85^k}}$

BOLT BEARING @ 1/4" ANGLE $= \dfrac{62^{k/in}}{\phi = 0.75} \times 0.25 \times 4 = \underline{\underline{83^K}}$

@ 5/16" ANGLE $= \underline{\underline{103^K}}$

BLOCK SHEAR @ L4x3x1/4 – $F_y = 44^{ksi}$ $F_u = 62^{ksi}$

GUSSET PL = 3/8"
∴ ANGLES CONTROL BLOCK SHEAR



$A_{NV} = \left(4.5'' - \tfrac{3}{2} \times (0.75 + 0.0625)\right) \times 0.25''$
$= 0.82 in^2$

$A_{NT} = \left(1.5'' - \tfrac{1}{2}(0.75 + 0.0625)\right) \times 0.25$
$= 0.27 in^2$

$R_n = 0.6 F_u A_{NV} + U_{BS} F_u A_{NT} \leq 0.6 F_y A_{gv} + U_{BS} F_u A_{NT}$

$U_{BS} = 0.5$ (ANGLE)

$= (0.6)(62)(0.82) + (0.5)(62)(0.27)$
$= 39^K$

$A_{GV} = 0.25 \times 4.5 = 1.125 in^2$

$0.6 F_y A_{gv} = 1.125 \times 0.6 \times 44 = 29.7^k$ ← CONTROLS

$0.6 F_u A_{NV} = 0.6 \times 62 \times 0.82 = 30.5$

$R_n = 38^k$ EA. ANGLE

$R_{n_{TOT}} = 2 \times 38^k = \underline{\underline{76^K}}$

Degenkolb

Subject:
Job:
Job Number:
Date:
By:
Section:
Checked By:
Page of

BLOCK SHEAR @ L5x3 1/2 x 5/16

$R \cong (76^k)(0.3125/0.25) = \underline{\underline{95^k}}$

GUSSET CONN w/ 2 BOLTS PER BM

$2 - 3/4"\phi$ BOLTS $= 42^k$

@ L4x3x1/4 $\quad T_{u_{BOLTS}} = 0.75\,T_{BRACE}$

$42^k = 0.75\,T_{BRACE}$

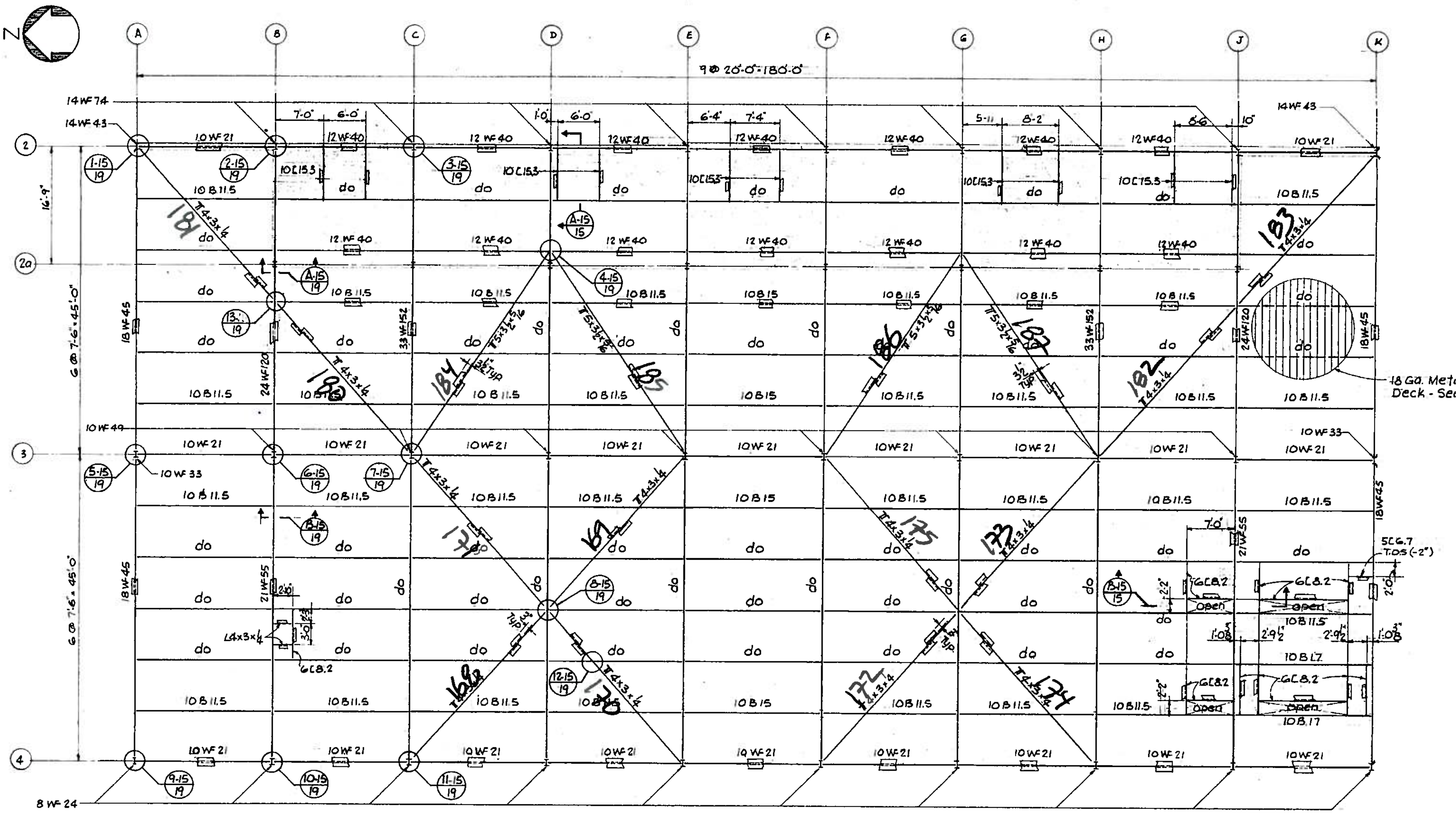
$T_n = \underline{\underline{56^k}}$

@ L5x3 1/2 x 3/16 - LINE 2a

$T_{u_{BOLTS}} = 0.55\,T_{BRACE}$

$42^k = 0.55\,T_{BRACE}$

$\underline{\underline{T_n = 76^k}}$

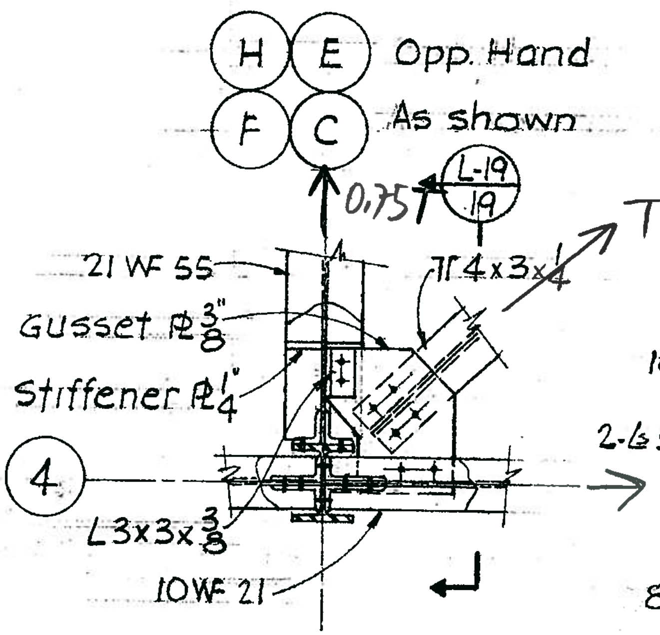


SAP ELEMENT #'s

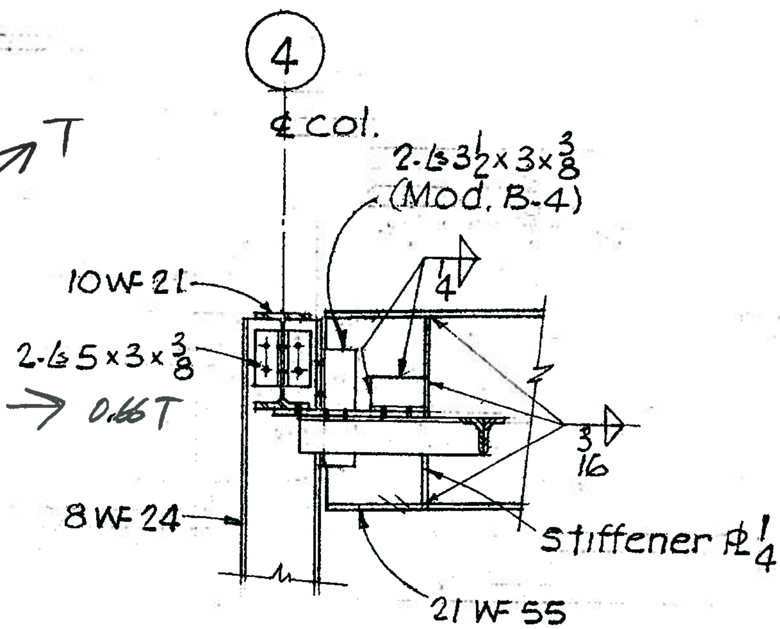
LOW ROOF FRAMING PLAN

Scale: 1/8"=1'-0"

Note: Shading indicates limits of



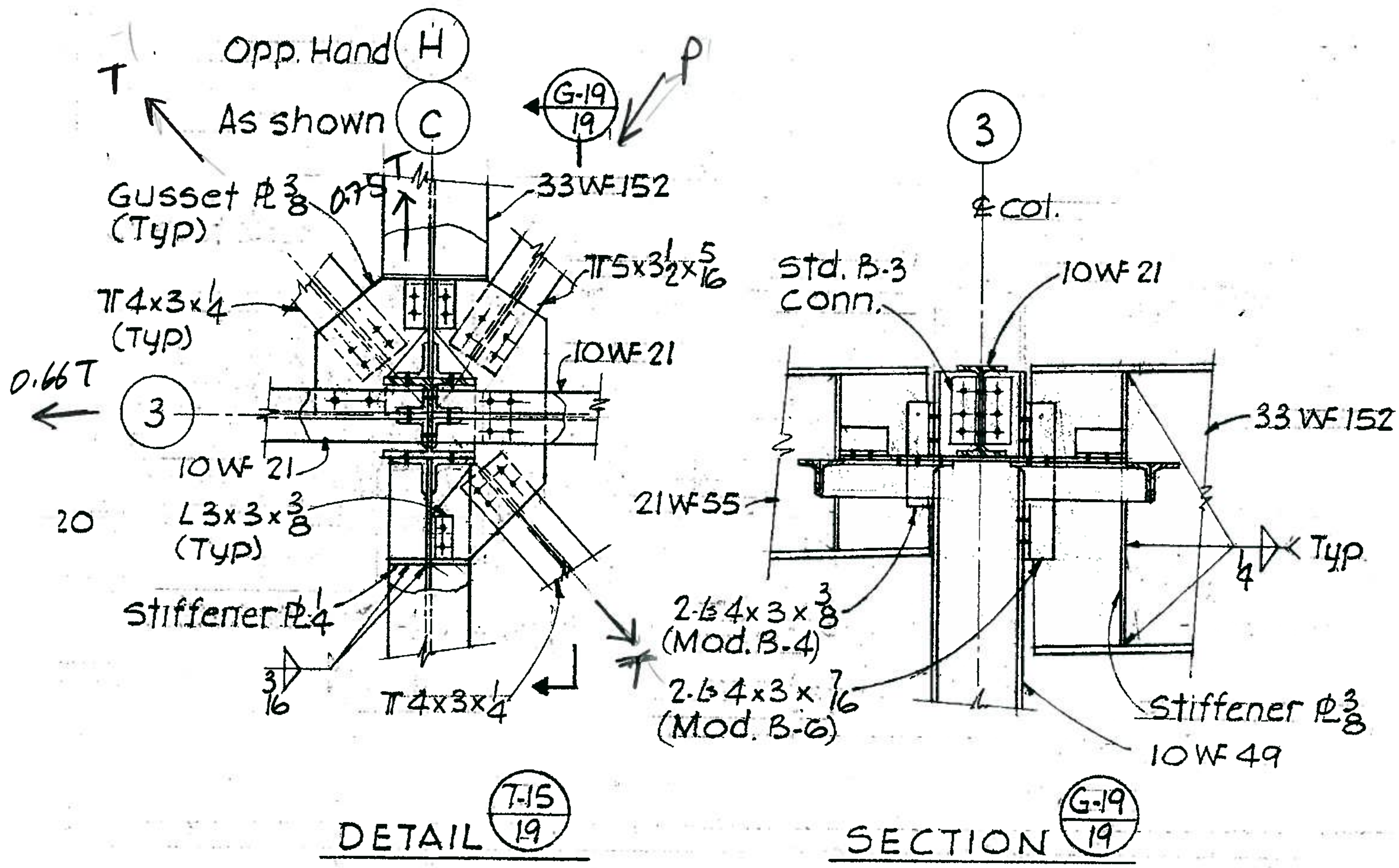
DETAIL 11-15/19



SECTION L-19/19

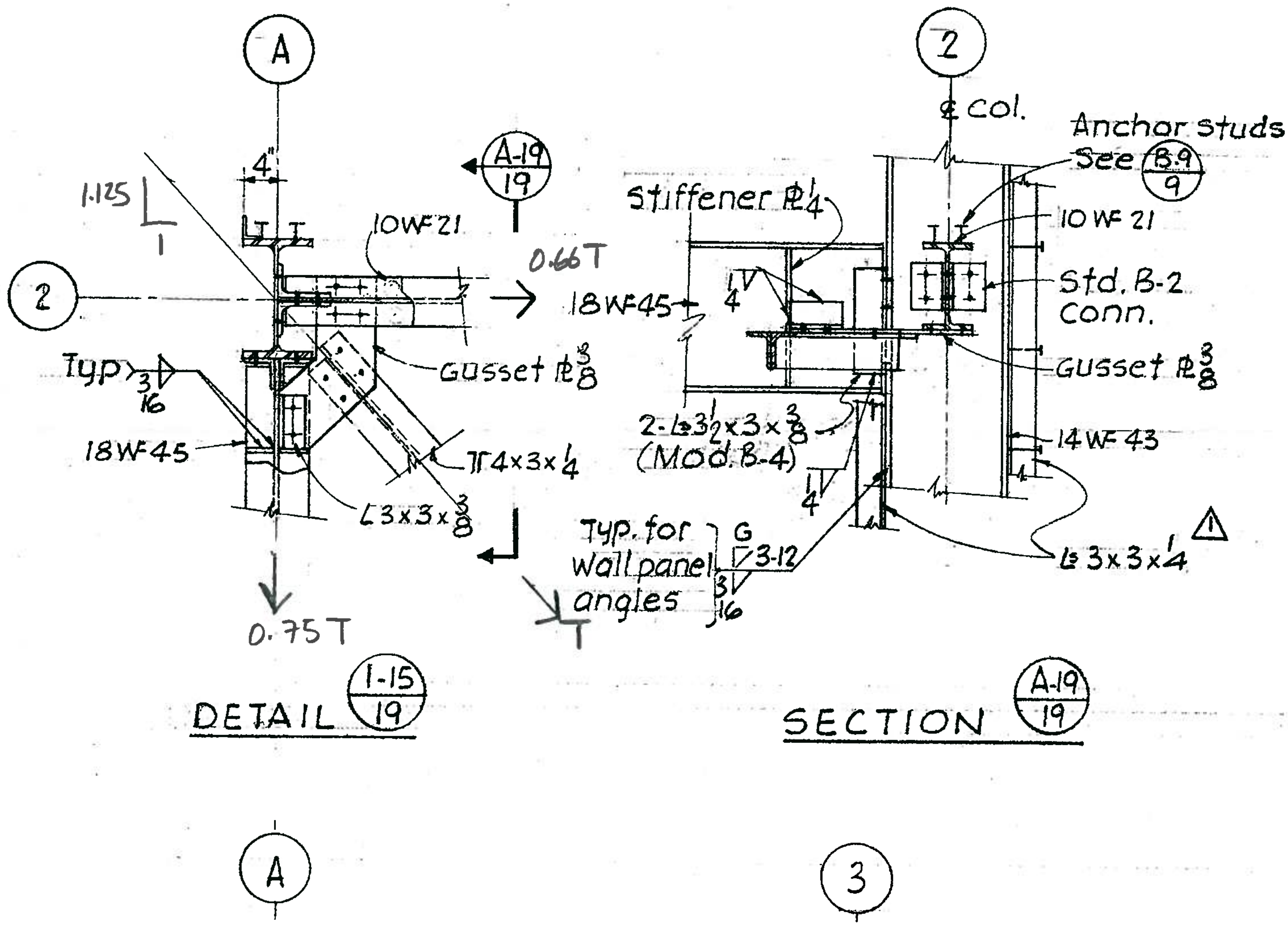
SAP ELEMENTS 168, 170, 172, 174

$T_n = 56^k$



SAP ELEMENTS 180 & 182

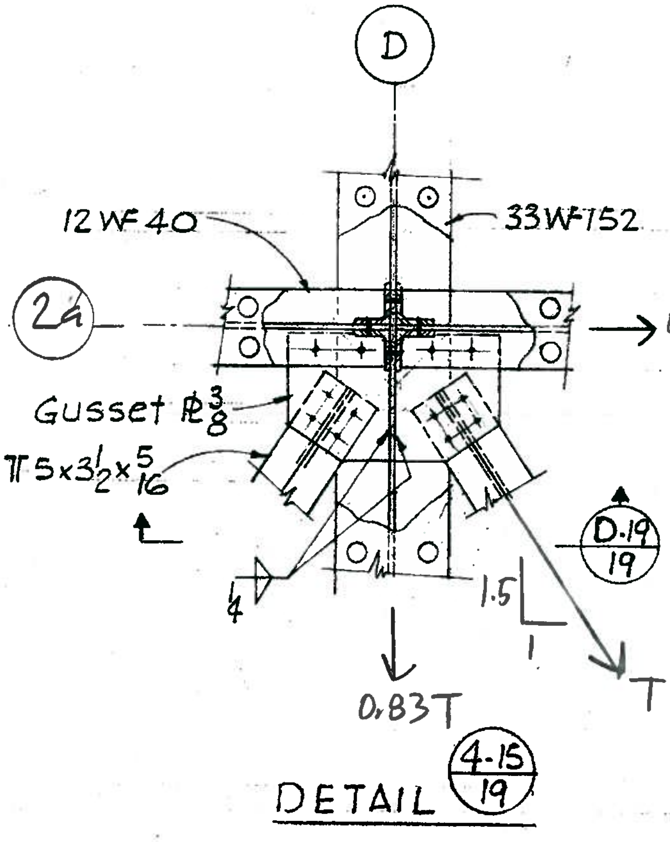
$T_n = 56^K$

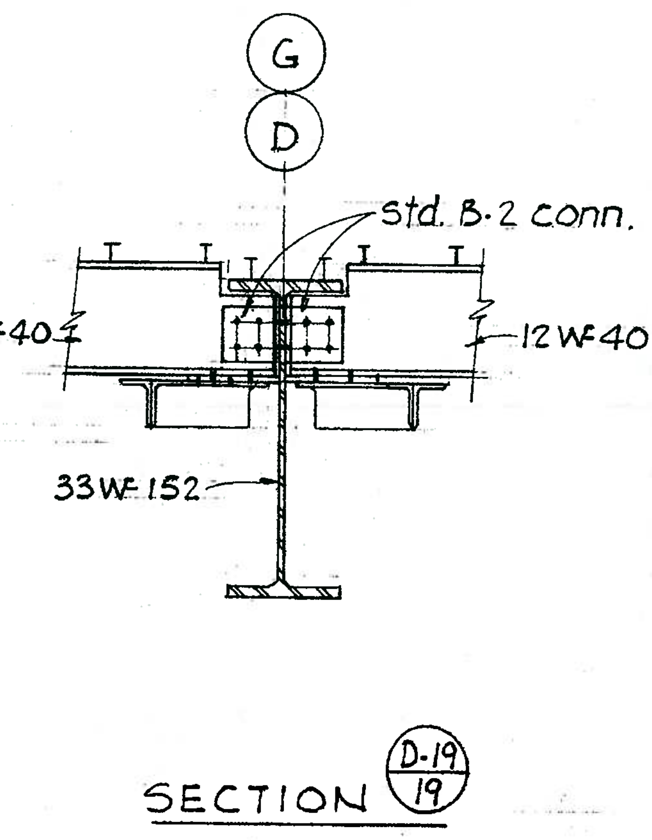


SAP ELEMENTS 181 & 183

$T_n = 56^k$

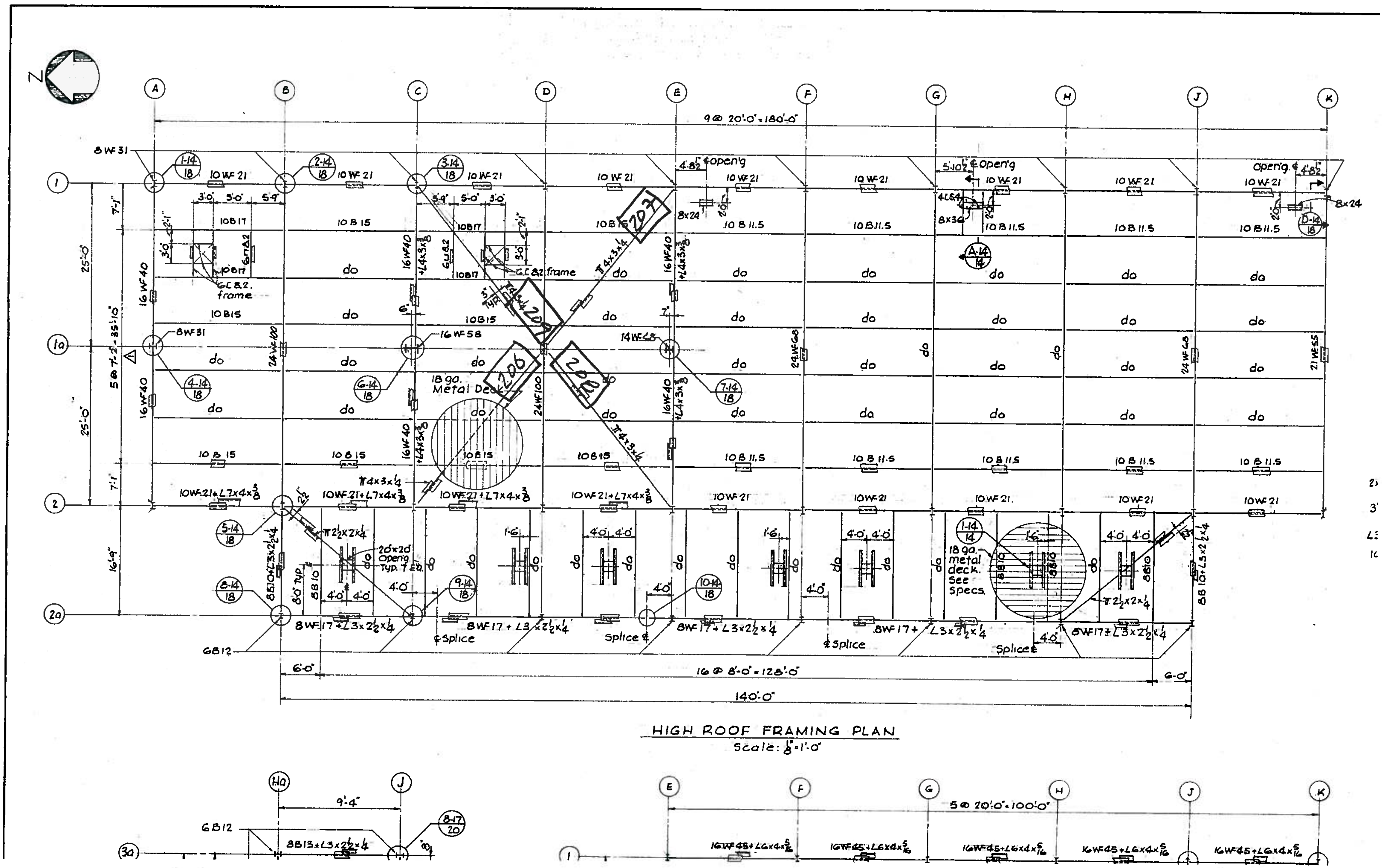
180 & 182 SIM.

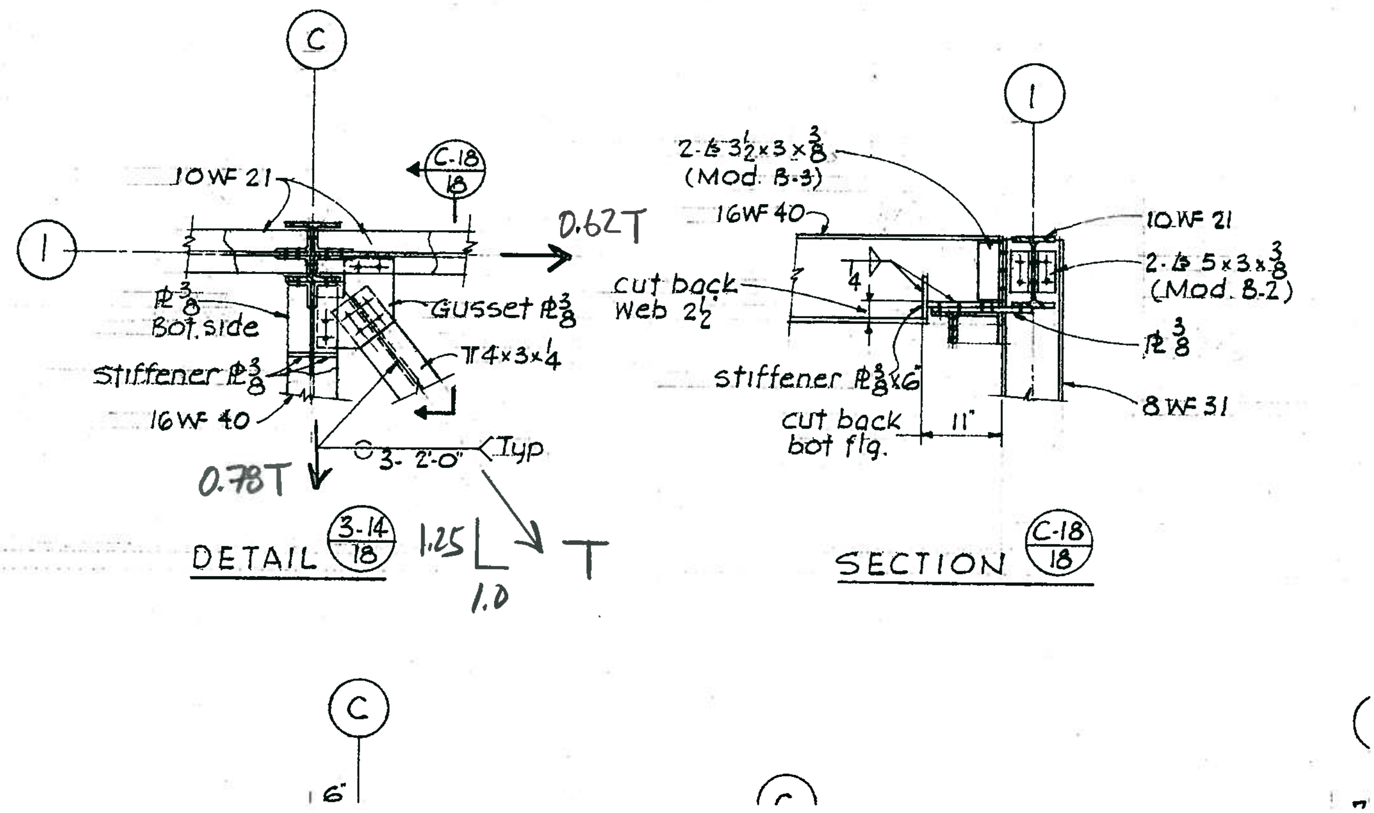




SAP ELEMENTS 184, 185, 186 & 187

$T_u = 76^K$


N
9 @ 20'-0" = 180'-0"
10 WF 21
10 B 15
10 B 11.5
do
16 WF 40
24 WF 100
24 WF 68
21 WF 55
14 WF 68
16 WF 58
8 WF 31
18 ga. Metal Deck
18 ga. metal deck. See specs.
10 WF 21 + L7x4x3/8
8 WF 17 + L3x2 1/2x1/4
8 B 10 + L3x2 1/2x1/4
20x20 Opng Typ. 7 ea.
Splice
GB 12
16 @ 8'-0" = 128'-0"
140'-0"
5 @ 7'-2" = 35'-10"
25'-0"
16'-9"
6'-0"
4'-0"
HIGH ROOF FRAMING PLAN
Scale: 1/8" = 1'-0"
9'-4"
8 B 13 + L3x2 1/2x1/4
5 @ 20'-0" = 100'-0"
16 WF 45 + L6x4x5/16
207
209
206
208



SAP ELEMENTS 206, 207, 208 & 209

$$T_n = 42^k/0.78 = \underline{\underline{54^k}}$$

Evaluation of Out-of-Plane Wall Strength and Anchorage

December 2013

Degenkolb

Subject:

Job:

Job Number: Date:

By: Section:

Checked By: Page of

OUT-OF-PLANE WALL STRENGTH

$F_p = 0.4 S_{XS} X W_p$ $\quad X = 1.3$

$S_{XS} = 0.98$

$F_p = (0.4)(0.98)(1.3) W_p$ $\quad F_{p\,min} = (0.1)(1.3)(W_p) = 0.13 W_p$

$= 0.51 W_p$

6" CONC. PANEL $W_p = 75\,psf$

$F_p = 38\,psf$

PANELS @ HIGH ROOF

$SPAN = (34'\text{-}6") - (1'\text{-}1") - (1'\text{-}9") = 31.7'$

$M_u = \dfrac{(38\,psf)(1')(31.7')^2}{8} = 4773\,\#\text{-}ft$

6" PANEL W/ #4 @ 12" O.C. EA. WAY

$M_n = (40^{ksi})(0.2\,in^2)\left(2.75" - \dfrac{(40)(0.2)}{(2)(0.85)(3)(12)}\right)$

$= 21^{k\text{-}in} = 1746\,\#\text{-}ft < M_u = 4773\,\#\text{-}ft$

NO GOOD DCR = 2.7

CHECK PANELS TO SPAN HOR. TO COLS

SHEAR: $V_u = 38\,psf \times 1' \times 31.7' \times \tfrac{1}{2} = 602\,\#$

$V_n = (2.75")(12")\,2\sqrt{3000} = 3615\,\# > 602\,\#$ OK

Degenkolb

Subject:

Job:

Job Number:

Date:

By:

Section:

Checked By:

Page of

PANELS @ LOW ROOF

$SPAN = (24'-0") - (1'-0") - (1'-9") = 21.25'$

$$M_u = \frac{(38psf)(1')(21.25')^2}{8} = 2145 \text{ \#-ft}$$

SAME PANEL REINF. AS @ HIGH ROOF

$M_n = 1746 \text{ \#-ft} < 2145 \text{ \#-ft}$ $\quad$ DCR = 1.23

DCR = 1.0 $\quad$ FOR $F_y = F_{ye} = 50^{ksi}$

SHEAR : $V_u = 38psf \times 21.25' \times 1' \times \frac{1}{2} = 404^{\#}$

$V_n = 3615^{\#} > 602^{\#}$ $\quad$ OK

CHECK PANELS TO SPAN BET. STL. STRONG BACKS

HOW FAR CAN PANELS SPAN?

$$M_n = 1746 \text{ \#-ft} = \frac{(38psf)(1')(L)^2}{8}$$

$L \leq 19'-2"$ $\quad$ (HOR. OR VERT.)

FOR EXPECTED STRENGTH, $F_y = 1.25 \times 40^{ksi}$
$= 50^{ksi}$

$M_n = 1.25 \times 1746 \text{ \#-ft} = (38)(L)^2/8$

$L \leq 21.4'$

Degenkolb

Subject:

Job:

Job Number:

By:

Checked By:

Date:

Section:

Page of

CHECK STEEL COLS/STRONG BACKS FOR PRECAST O-O-P WALL FORCES

W8x31 – LINE 1 $F_y = 44^{ksi}$

SPAN = 31' − 0.66' = 30.33'

TRIB WIDTH = 20'

$w = (20')(38\,psf) = 760\,plf$

$M_u = \frac{(760\,plf)(30.33')^2}{8} = 87409^{\#\text{-}ft} = 1049^{k\text{-}in}$

$P_u = 1.1\,(24\,psf + 20\,psf/4)(20' \times 25') = 16^k$

$Kl = 30.33'$ (BUCKLING) $Kl/r_x = 105$

$L_B = 16'$ (BENDING)

$P_n = 9.12 \times 19.4^{ksi} \times \frac{1}{0.9} = 197^k$

$M_n \cong 93^{k\text{-}ft}/0.9 \times 44/50 = 91^{k\text{-}ft} = 1092^{k\text{-}in}$

$DCR = \frac{16^k}{(2)(197^k)} + \frac{1049^{k\text{-}in}}{1092^{k\text{-}in}} = \underline{1.0 \quad OK}$

W8x31 – LINE 1a & A

TRIB WIDTH = 25'

SPAN = 31' − 1.33' = 29.66'

$M_u = (25')(38\,psf)(29.66')^2/8 = 104466^{\#\text{-}ft} = 1253^{k\text{-}in}$

$P_u = 1.1\,(29\,psf)(25' \times 10') = 8^k$

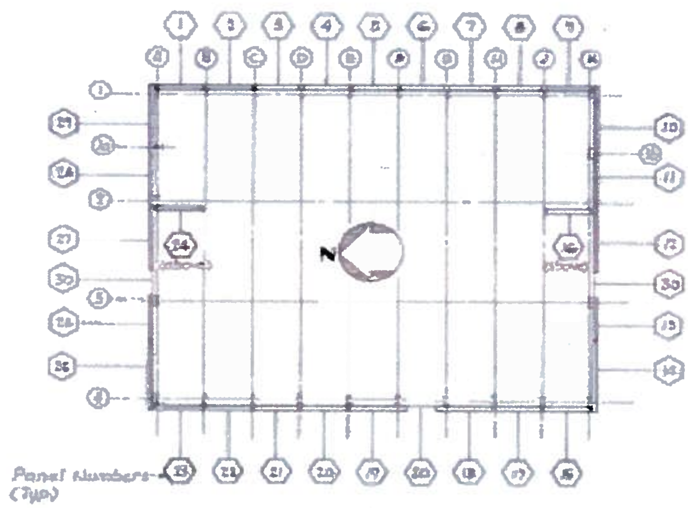
$DCR = \frac{8}{(2)(197)} + \frac{1253}{1092} = \underline{1.17 \quad NG}$

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject: Precast Panel - Out-of-plane strength (span check) **Job Number:** B3189012.00 **Date:** 12.3.13
Job: LLNL B341 Increment I **By:** AMN **Section:**
Checked By:

Panel Number	Horizontal Span (ft)	Vertical Span (ft)	Check Span < 19'-2"	Check Span < 21'-5"
1	19.0	16.25	OK	OK
2	19.0	16.25	OK	OK
3	19.0	16.25	OK	OK
4	19.0	16.25	OK	OK
5	19.0	18.20	OK	OK
6	19.0	18.20	OK	OK
7	19.0	18.20	OK	OK
8	19.0	18.20	OK	OK
9	19.0	18.20	OK	OK
10	24.5	18.25	OK	OK
11	24.5	18.25	OK	OK
12	45.0	21.25	NO	OK
13	45.0	20.75	NO	OK
14	45.0	20.58	NO	OK
15	19.0	N/A	OK	OK
16	19.0	10.50	OK	OK
17	19.0	20.25	OK	OK
18	19.0	20.25	OK	OK
19	19.0	20.25	OK	OK
20	19.0	20.25	OK	OK
21	19.0	20.25	OK	OK
22	19.0	20.25	OK	OK
23	19.0	20.25	OK	OK
24	19.0	10.50	OK	OK
25	45.0	20.58	NO	OK
26	45.0	20.75	NO	OK
27	45.0	21.25	NO	OK
28	24.5	18.25	OK	OK
29	24.5	18.25	OK	OK
30	14.0	22.00	OK	OK



Degenkolb

Subject:

Job:

Job Number: | Date:

By: | Section:

Checked By: | Page of

SUMMARY OF OUT-OF-PLANE STRENGTH

LINE 1 : PANELS SPAN TO COLS / MEZZ SLAB OK

LINE 4 ; PANELS SPAN TO ROOF DIAPHRAGM OK

LINE A & K LOW ROOF : PANELS SPAN TO ROOF OK

LINE A @ HIGH ROOF : PANELS SPAN TO COL STRONG BACK COLS NG

LINE K @ HIGH ROOF : PANELS SPAN TO MEZZ & HIGH ROOF OK

PRECAST PANEL OUT-OF-PLANE WALL ANCHORAGE

ANCHORAGE DEMANDS

$F_p = 0.4 S_{XS} K_a K_h \chi W_p$

$\chi = 1.3$ LS

$S_{XS} = 0.98$

$K_a = 1.0 + \frac{L_f}{100}$, $L_f = 100'$

$K_a = 2.0$

$K_h = 1.0$ (FLEXIBLE DIAPHRAGM)

$F_p = (0.4)(0.98)(2)(1)(1.3) W_p$

$\underline{= 1.02 W_p}$ ⇐ CONTROLS

$F_{p_{min}} = 0.2 K_a \chi W_p = (0.2)(2)(1.3) W_p = 0.52 W_p$

$W_p = 75$ psf − 6" CONC.

$\underline{F_p = 77 \text{ psf}}$

Degenkolb

Degenkolb Engineers
1300 Clay Street, 9th Floor
Oakland, California

Subject: Precast Panel - Out-of-plane Ancorage | **Job Number:** B3189012.00 | **Date:** 12.3.13
Job: LLNL B341 Increment I | **By:** AMN | **Section:**
Checked By:

Fp = 77 psf

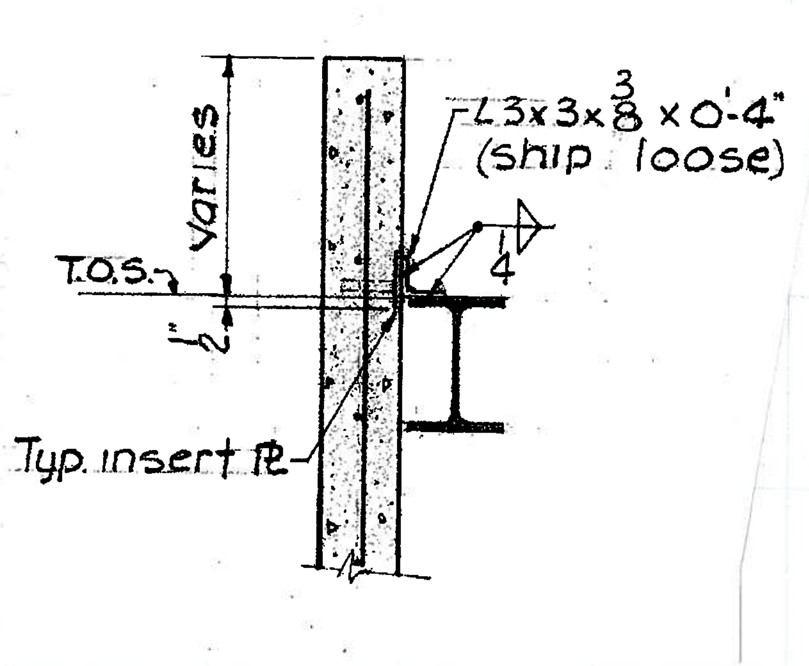
Beam Connection Capacity = 4.1 kips
Column Connection Capacity = 3.8 kips

Panel Number	Horizontal Trib Width(ft)	Vertical Trib Width (ft)	Vertical Insert Spacing (ft)	Horizontal Insert Spacing (ft)	Tension Demand at Col Anchorage	Tension Demand at Beam Anchorage	Col. Anchorage DCR	Beam Anchorage DCR
1	10.0	N/A	3	3	2.3	N/A	0.61	N/A
2	10.0	N/A	3	3	2.3	N/A	0.61	N/A
3	10.0	N/A	3	3	2.3	N/A	0.61	N/A
4	10.0	N/A	3	3	2.3	N/A	0.61	N/A
5	10.0	N/A	3	3	2.3	N/A	0.61	N/A
6	10.0	N/A	3	3	2.3	N/A	0.61	N/A
7	10.0	N/A	3	3	2.3	N/A	0.61	N/A
8	10.0	N/A	3	3	2.3	N/A	0.61	N/A
9	10.0	N/A	3	3	2.3	N/A	0.61	N/A
10	N/A	12.50	3	3	N/A	2.9	N/A	0.70
11	N/A	12.50	3	3	N/A	2.9	N/A	0.70
12	N/A	12.50	3	3	N/A	2.9	N/A	0.70
13	N/A	12.50	3	3	N/A	2.9	N/A	0.70
14	N/A	12.50	3	3	N/A	2.9	N/A	0.70
15	10.0	N/A	3	3	2.3	N/A	0.61	N/A
16	10.0	10.50	3	3	2.3	2.4	0.61	0.59
17	N/A	10.13	3	3	N/A	2.3	N/A	0.57
18	N/A	10.13	3	3	N/A	2.3	N/A	0.57
19	N/A	10.13	3	3	N/A	2.3	N/A	0.57
20	N/A	10.13	3	3	N/A	2.3	N/A	0.57
21	N/A	10.13	3	3	N/A	2.3	N/A	0.57
22	N/A	10.13	3	3	N/A	2.3	N/A	0.57
23	N/A	10.13	3	3	N/A	2.3	N/A	0.57
24	10.0	10.50	3	3	2.3	2.4	0.61	0.59
25	N/A	12.50	3	3	N/A	2.9	N/A	0.70
26	N/A	12.50	3	3	N/A	2.9	N/A	0.70
27	N/A	12.50	3	3	N/A	2.9	N/A	0.70
28	N/A	16.00	3	3	N/A	3.7	N/A	0.90
29	N/A	16.00	3	3	N/A	3.7	N/A	0.90

Degenkolb

Subject:
Job:
Job Number:
Date:
By:
Section:
Checked By:
Page of

TYPICAL PANEL CONN @ ROOF & STRONGBACK BMS



WELD TO BM

WALL TO BM GAP $= \dfrac{8'' - 5.75''}{2} = 1.125''$

WELD LENGTH $= 3'' - 1.125'' = 1.875''$ EA. SIDE

$A_{WELD} = (2)(1.875'') = 3.75 \ in^2/in$

$S_{WELD} = \dfrac{(2)(1.875'')^2}{6} = 1.17 \ in^3/in$

$\sigma_V = V_u/3.75 \qquad \sigma_B = V_u \times 1.5''/1.17$

$\sigma_R = \sqrt{(V_u/3.75)^2 + (1.5V_u/1.17)^2} = 4 \times 1.39/0.75 = 7.4^{k}/in$

$\underline{\underline{V_u = 5.7^{k}}}$

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

BENDING OF ANGLE

$$M_u = 1.5'' \times V_u = M_n$$

$$M_n = \frac{(0.375'')^2 (4'')}{4} \times 44^{ksi} = 6.2^{k\text{-}in}$$

$$V_u = 6.2^{k\text{-}in} / 1.5'' = \underline{\underline{4.1^k}}$$

WELD TO ANGLE

$$V_n = (2)(3'')(1.39/0.75)(4) = \underline{\underline{44^k}}$$

CONCRETE PANEL INSERT

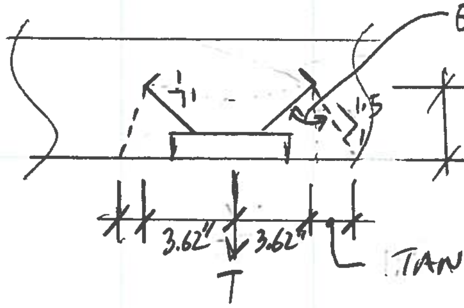
STRAP CAPACITY IN TENSION

$$T_n = \frac{(2)(3/16'')(1'')(62^{ksi})}{\sqrt{2}} = \underline{\underline{16^k}}$$

CONCRETE ANCHORAGE

EMBED = 2.625"

$\theta = 90 - \tan^{-1}(1/1.5) = 56°$



$\tan(11.3°) = \frac{X}{2.625''} \Rightarrow X = 0.5''$

$$A_{NC} = (2)(3.62'' + 0.5'')(3 \times 2.625'') = 65\ in^2$$

$$A_{NCO} = (9)(2.625'')^2 = 62\ in^2$$

$$N_b = (24)(1)\sqrt{4000}\,(2.625)^{1.5} = 6455^{\#}$$

Degenkolb

Subject:

Job:

Job Number: Date:

By: Section:

Checked By: Page of

$$N_{cb} = \frac{A_{NC}}{A_{NCO}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$

$\Psi_{ec,N} = 1.0$ $\Psi_{ed,N} = 1.0$ $\Psi_{c,N} = 1.25$ (NO CRACKING)

$\Psi_{cp,N} = 1.0$

REDUCE FURTHER 0.75 FOR HIGH SEISMIC

$$N_{cb} = (0.75) \frac{65 in^2}{62 in^2} \times 1 \times 1 \times 1.25 \times 1 \times 6455^{\#}$$

$$\underline{\underline{N_{cb} = 6344^{\#}}}$$

ANGLE BENDING CONTROLS

$$\underline{T_n = 4.1^k}$$

SHARE BENDING BET. ANGLE & INSERT

$e = 0.75''$

ANGLE CAPACITY $= 8.2^k$

RECALC ECCENTRICITY FOR CONC. ANCH.

$$\Psi_{ec,N} = \frac{1}{1 + \frac{2 \times 0.75''}{3 \times 2.625}} = 0.84$$

$$N_{cb} = 6344^{\#} \times 0.84 = 5328^{\#}$$

ANCHOR CONTROLS $\Rightarrow$ $\underline{\underline{T_n = 5.3^k}}$

Degenkolb

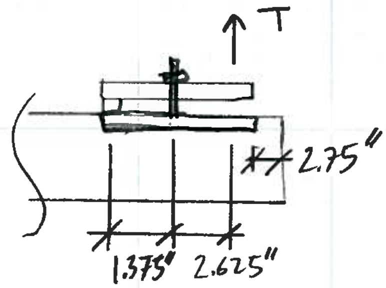
Subject:
Job:
Job Number:
Date:
By:
Section:
Checked By:
Page of

CONN @ COL. ON LINES 1 & 4

WELD (PL TO COL FLNG)

$T_n = 2'' \times 1.39/0.75 \times 3 \times 2 = \underline{\underline{22^K}}$

3/4"Ø STUD



$T_{n\,STUD} = 0.44 in^2 \times 45^{ksi}$
$= 19.8^K$

$T_{u\,STUD} = \dfrac{T \times (1.375 + 2.625)}{1.375} = 19.8^K$

$\underline{\underline{= 6.8^K}}$

CONCRETE ANCHORAGE

SIMILAR TO INSERT @ BM

$N_B = 6455^{\#}$ $\quad A_{NC} = 65 in^2$ $\quad A_{NCO} = 62 in^2$

$\Psi_{ec,N} = \dfrac{1}{\left(1 + \dfrac{2 \times 2.625}{3 \times 2.625}\right)} = 0.6$ (ECCENTRICITY)

$\Psi_{ed,N} = 1 \quad \Psi_{c,N} = 1.25 \quad \Psi_{cp,N} = 1.0$

$N_{CB} = 0.75 \times \dfrac{65}{62} \times 0.6 \times 1 \times 1.25 \times 1 \times 6455^{\#}$

$\underline{\underline{N_{cb} = 3.8^K}}$

CONCRETE CONTROLS

$\underline{\underline{T_n = 3.8^K}}$

Degenkolb

Subject:

Job:

Job Number:

By:

Checked By:

Date:

Section:

Page of

CHECK CROSS-TIES @ LOW ROOF

EAST-WEST LINES

SPACING = 20'

WALL ANCH. FORCE = 12' x 77 psf = 924 plf

$T_U = 20' \times 924\,plf = 18.5^k$

21WF55

$T_n = 44^{ksi} \times 16.2\,in^2$

$= 713^k$

$M_u = \dfrac{1.1(24 + 20/5)(20')(45)^2}{8} = 1938^{k-in}$

$M_n = 44^{ksi} \times 126\,in^3 = 5544^{k-in}$

$DCR = \dfrac{18.5^k}{(2)(713^k)} + \dfrac{1938^{k-in}}{5544^{k-in}} = \underline{0.35 \text{ OK}}$

CONN = 2L4x3x3/8 w/ 4-3/4"Ø HSB EA. LEG

OK BY INSP.

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

NORTH-SOUTH LINES

SPACING $= 7.5'$

$T_u = \frac{7.5'}{20'} \times 18^k = 7^k$

10B11.5

$T_n = 44^{ksi} \times 3.38_{in}{}^2 = 149^k$

DCR DUE TO TENSION ONLY $< 5\%$ OK BY INSP.

CONN 2 - 5/8" ϕ HSB & 2-L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$ OK BY INSP.

OK

Degenkolb

Subject:	Job Number:	Date:
Job:	By:	Section:
	Checked By:	Page of

CHECK CROSS TIES @ HIGH ROOF

EAST – WEST LINES

$$T_u = (20')(77psf \times 31'/2) = 24^k$$

$$T_n = 20.1 in^2 \times 44^{ksi} = 884^k \quad 24WF68$$

OK BY INSP. DCR DUE TO T_u 0.027

NORTH–SOUTH LINES

SPACING = 7.17'

$$T_u = \frac{7.17'}{20'} \times 24^k = 8.6^k$$

10B11.5 $T_n = 149^k$

DCR DUE TO TENSION = 0.057 OK BY INSP.